Slope Stability in Road Construction

U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF LAND MANAGEMENT
OREGON STATE OFFICE
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SLOPE STABILITY IN ROAD CONSTRUCTION

A GUIDE TO THE CONSTRUCTION OF STABLE ROADS IN WESTERN OREGON AND NORTHERN CALIFORNIA

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TABLE OF CONTENTS

INTRODUCTION

There is no question that road construction is an extremely important activity in western Oregon and northern California. Many thousands of man hours are expended each year to locate, design, construct and maintain road systems. The construction of stable roads can sometimes be difficult because of the steep terrain, weak geologic material, and the heavy winter rainfall that is common to the coastal mountains. Road failures can exert a tremendous impact on natural resources and can cause serious economic losses because of blocked streams, degraded water quality, destroyed bridges, ruined spawning sites, lowered productivity of forest lands, and damage to private property. It is vital that those personnel engaged in road building activities be aware of the basic principles of slope stability and understand how these principles can be used to construct stable roads through various geologic materials with specific conditions of slope and soil.

This booklet presents a general outline of the geology of western Oregon and northern California to provide a background for discussion of specific problems of road construction. Basic slope stability is illustrated by a description of the balance of forces that exists in undisturbed slopes, how these forces change as the road is constructed, and how ground water affects slope stability and causes road failures. The final section of this publication discusses techniques for constructing stable roads on specific geologic materials and soils. Several appendices are included which discuss certain concepts in soil mechanics in greater detail together with basic techniques in slope stability analysis for those readers who desire this information.

These techniques represent a summary of the knowledge of engineers, foresters, geologists, and soil scientists from the Bureau of Land Management, Forest Service, private industry, and the universities. The techniques presented here are not meant to be the ultimate in technology but merely the "state of the art" at this time. The users of this publication are encouraged to revise and add to these techniques for construction of stable roads as their experience increases. This guide, if kept up to date, can provide an introduction to stable road construction for new employees or transferrees who are unfamiliar with regional roadbuilding problems.

GEOLOGY OF WESTERN OREGON AND NORTHERN CALIFORNIA

Western Oregon and northern California have a complex geologic history of volcanic activity, deposition of thick beds of sedimentary material, and more volcanic activity with erosion operating continuously at varying rates. Figure 1 shows the four physiographic provinces that illustrate this varied geologic history: the Western Cascades, the Coast Range, the Klamath Mountains, and the Willamette Valley. The relatively level portions of the Willamette Valley will be excluded from any further consideration in this publication because road construction within the valley entails only a slight risk of slope failure.

The wide variety of geologic materials present within the subject area of this guide can be separated into major groups (Figure 2), each with particular characteristics that affect road location, design, construction, and maintenance. When Figures 1 and 2 are compared it is seen that the Coast Range is composed mostly of bedded sediments with scattered intrusions of volcanic material. The Western Cascade province is composed mostly of volcanic materials with small isolated areas of bedded sediments and granitic rock. The Klamath Mountains province is the most geologically complex with very old sedimentary and volcanic rocks, locally metamorphosed (altered by heat and pressure), and with intrusions of granite and serpentine.

It should be emphasized that the simple description of the geology of western Oregon and northern California that follows has several purposes: it illustrates the variety and regional variability of geologic materials; it introduces basic terms and concepts in geology; and finally, it provides a background for the interpretation of detailed geologic maps that may be consulted for a specific project.

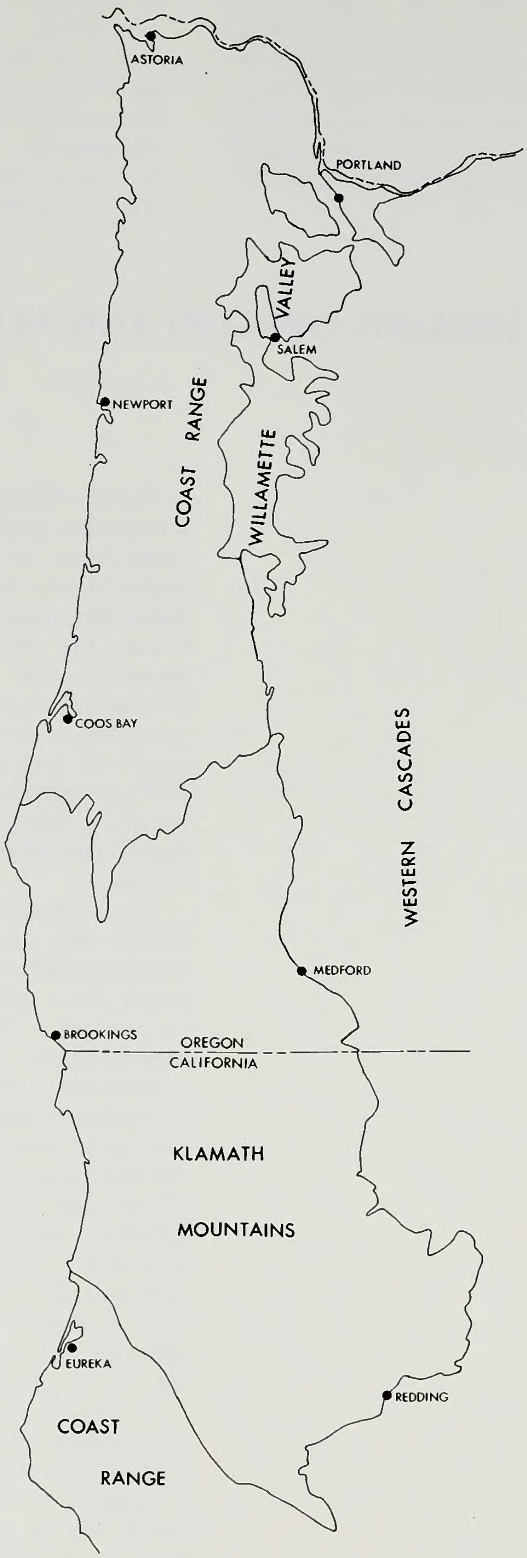


Figure 1. Physiographic Provinces of Western Orgeon and Northern California

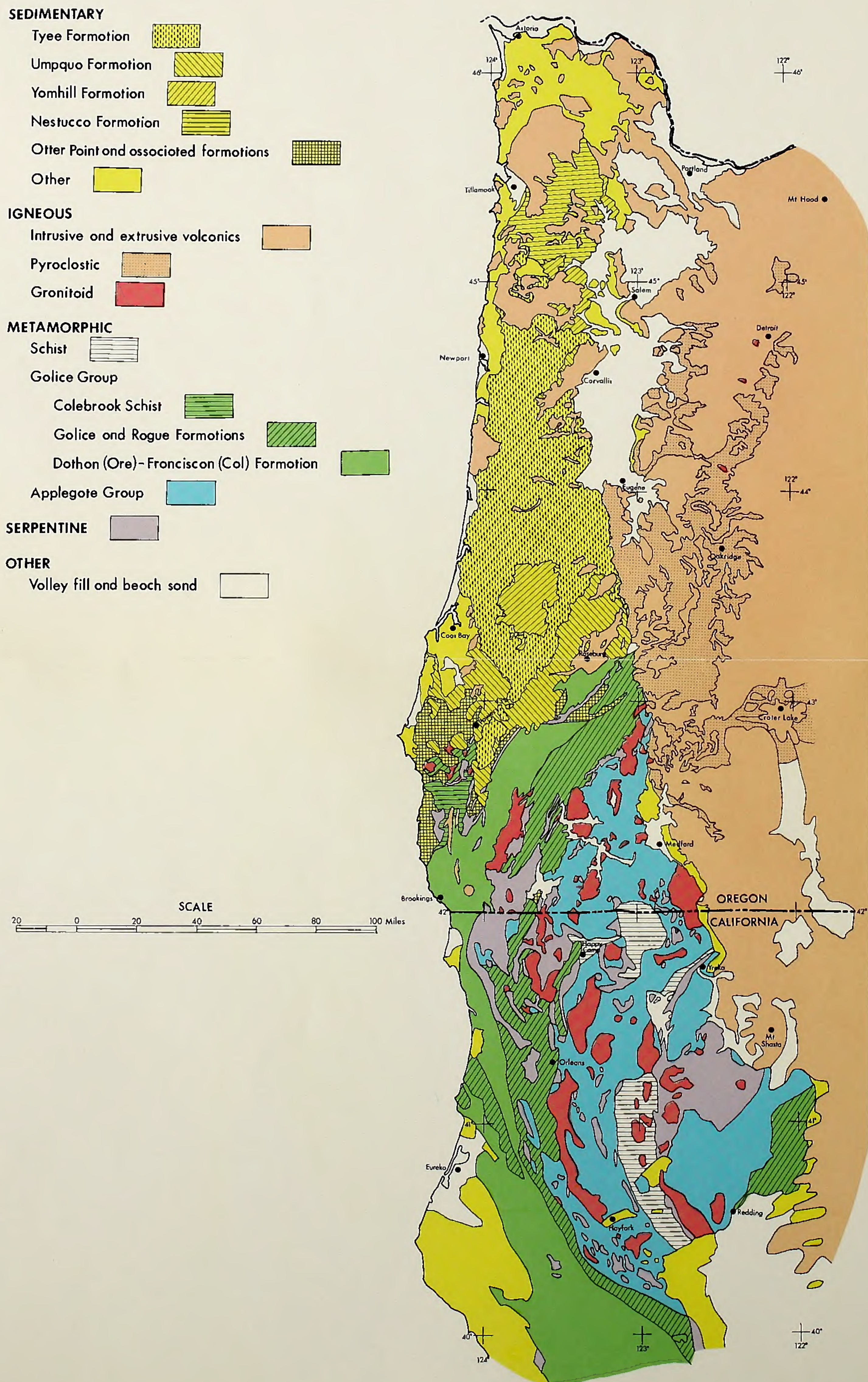


Figure 2. Geologic Map of Western Oregon and Northern California

CHARACTERISTICS OF COMMON GEOLOGIC MATERIALS (Adapted from Baldwin; Wells and Peck)

1. Bedded Sediments

A sedimentary rock is composed of fragments that have been deposited from a transporting medium, such as water, ice, or air. Sedimentary rocks are usually deposited in layers known as *beds* or *strata*, that are separated by *bedding planes*. Bedding is indicated by changes in particle size or color. The beds vary in thickness and are essentially parallel although they may be warped or tilted from the horizontal by uplift, folding, and faulting.

The bedded sediments may include layers of fine-grained fragmental rocks such as *shale*, *mudstone*, and *siltstone*, that may be interbedded between thicker layers of medium-grained rocks such as *sandstone*. *Graywacke* (pronounced "gray-wacky") is a variety of dark colored sandstone with angular grains of quartz, feldspar, and small rock fragments set in a matrix of fine particles. Sediments are called marine if they were deposited in the sea or estuaries, and nonmarine or continental if they were deposited in fresh water. Sediments are compacted by the weight of overlying water and sediments and are cemented to varying degrees of hardness by minerals precipitated from ground water. Flood waters, glacial melt, or shore waves may deposit sand, gravel, cobbles and boulders into poorly sorted masses of material. When this material is covered with other deposits and saturated with waters rich in minerals, a *conglomerate* is formed. This material may be weakly or strongly cemented and the texture tends to vary both vertically and laterally. The cementing material may be removed by weathering processes when conglomerate deposits are exposed either by natural erosion or by road construction.

A group of beds of sedimentary rock that are distinctive enough to be described and mapped as a unit is called a *formation*, and is given a formational name. Five formations of bedded sediments that occupy the greatest area in western Oregon are the *Tyee*, *Yamhill*, *Umpqua*, *Otter Point*, and *Nestucca*.

The Tyee occupies the greatest area of any formation in western Oregon (see Figure 2). The Tyee has massive beds, each of which grades upward from sandstone to siltstone. The colors range from medium-gray to bluish-gray on fresh surfaces and may appear tan to gray on weathered surfaces (Figures 3 and 4).

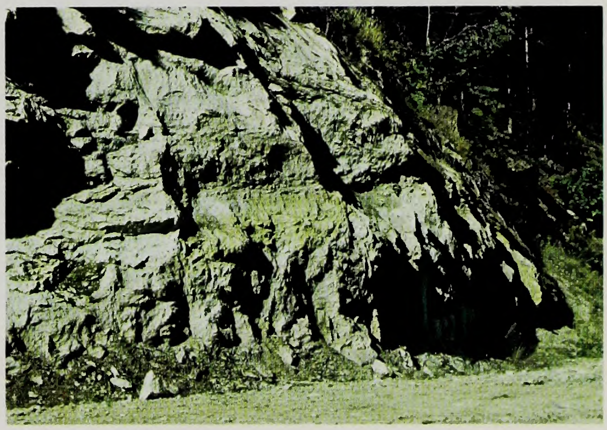


Figure 3. Sandstone from the Tyee Formation

Figure 4. Tyee Formation showing massive beds

Figure 5. Thin beds of mudstone in the Yamhill formation

Figure 6. Pebbly conglomerate from the Umpqua formation

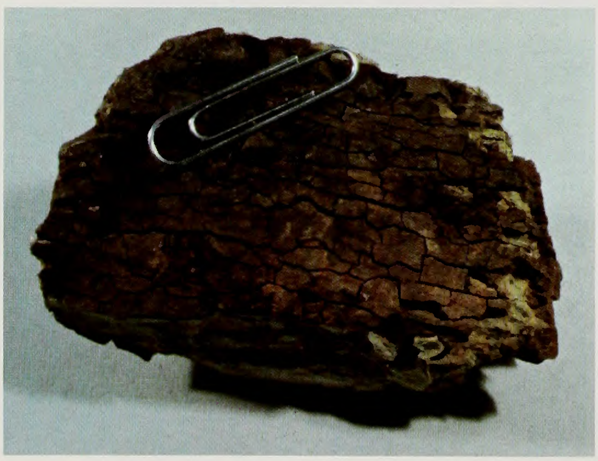


Figure 7. Sample of siltstone material

The Yamhill Formation contains gray to greenish-gray massive sandstone beds with thinner layers of medium-gray to dark-gray mudstone, siltstone, or shale (Figure 5). The Yamhill Formation is found in two general areas (Figure 2), one west of Salem and another northeast of Coos Bay.

The Umpqua is a complex formation with three major units or subdivisions which are presently being considered for separation into three distinct formations (Baldwin, 1973). The lowest, or oldest, unit (proposed Roseburg Formation) contains some volcanic rock in addition to beds of sedimentary rock. This material may be seen along Highway 42 between Coquille and Myrtle Point and also around Roseburg. The middle unit (proposed Lookingglass Formation) grades upward from thick layers of conglomerate through sandstone to siltstone; this material may be seen between Tenmile Creek and Lookingglass Creek southwest of Roseburg. The upper, or youngest, unit (proposed Flournoy Formation) is composed of pebbly conglomerate (Figure 6) that grades upward into siltstone and is found from Camas Valley almost to Dutchman Butte, southwest of Roseburg. Some portions of the Umpqua Formation may be darker than either the Tyee or Yamhill Formations; the layers of siltstone, in particular, tend to be very dark.

The Nestucca Formation covers a large area northwest of Salem and is composed of interbedded siltstone (Figures 7 and 8), claystone, and sandstone. In many places the Nestucca Formation is interbedded with basalt flows.

The appearance of the Otter Point Formation, and associated formations with similar characteristics, is that of highly sheared, tan, dark-gray or black sandstone with interstratified thin mudstone, minor conglomerate, and volcanic material (Figure 9). This material occurs in small areas from Myrtle Creek (south of Roseburg) on west to a large block along the southwest Oregon coast.

The remainder of the bedded sediments is composed of scattered formations which have geologic characteristics and slope stability problems similar to the sedimentary formations already discussed.

Figure 8. Siltstone material of the Nestucca Formation

Figure 9. The Otter Point formation contains sandstone, mudstone and shale.

2. Igneous Rocks (Exclusive of Granitoid Rock)

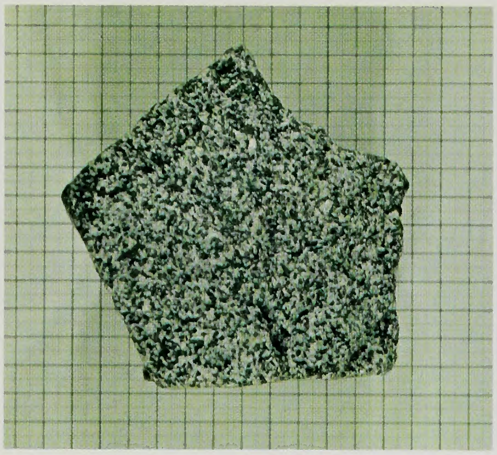
Igneous rocks may be divided into those that have cooled from molten masses beneath the surface of the earth, called *intrusive* rocks, and those that have flowed or been forcibly extruded upon the surface to cool, called *extrusive* rocks.

Intrusive igneous rocks are found in large, deep-seated irregular masses called *batholiths*, in smaller masses known as *stocks*, in tabular bodies known as *sills* that lie parallel to enclosing bedded sedimentary rocks, and in tabular masses known as *dikes* that cut across the enclosing strata. Some of the smaller dikes and sills cool rapidly into fine-grained rock. Many of the prominent peaks and ridges in the Coast Range of Oregon are the result of intrusions of igneous material into sedimentary rock where erosion has removed the softer sedimentary material to expose the harder igneous rock.

The western portion of the Cascade Range is underlain by thick layers of relatively hard extrusive igneous rocks, such as *basalt* and *andesite*, that are exposed extensively in the northern portion and along the higher elevations to the south. Some extrusive flows take place beneath water where the lava cools into rounded masses commonly 2 to 3 feet across. These rocks are referred to as *marine basalt* or *pillow lavas* because of their rounded shape (Figure 10) and these tend to be relatively soft and easily eroded. This material is common in a large block of igneous rock in the Coast Range northeast of Tillamook (Figure 2).

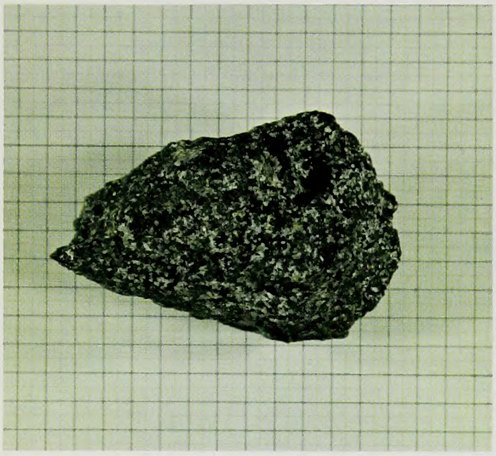
Photographs of four common extrusive and intrusive rocks are shown in Figure 11.

Figure 10. Rounded shapes of pillow lava

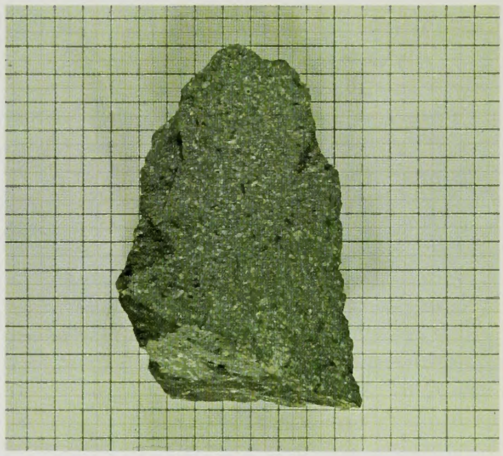


DIORITE

BASALT



GABBRO



ANDESITE

Figure 11. Four common intrusive and extrusive igneous rocks

3. Pyroclastic Rocks

Pyroclastic rock is a type of extrusive igneous rock that was expelled from volcanic vents. This material is partially molten when it is expelled and the individual pieces may fuse together to form a weak, porous rock. More commonly, the angular fragments are deposited with *volcanic ash* to form *volcanic breccia* (pronounced "breshia") that appears as coarse, angular fragments ¼ inch to 2 inches in size within a matrix of fine volcanic ash (Figure 12). If the size of the fragments imbedded in the ash are smaller than ¼ inch, then the resulting pyroclastic rock is known as *tuff* (pronounced "tough"); a photograph of this material is shown in Figure 13.

The strength and hardness of pyroclastic rocks depend to a large extent upon the amount of fusion and compaction that takes place between particles at the time of deposition. Generally, pyroclastic rocks are relatively soft and the volcanic ash matrix weathers rapidly to clay. Various kinds of ash, tuff and breccia are found over large areas on the slopes of the Cascades, and as inclusions within bodies of extrusive igneous rock in the Coast Range and the Klamath Mountains.

Figure 12. Red and green colored breccia

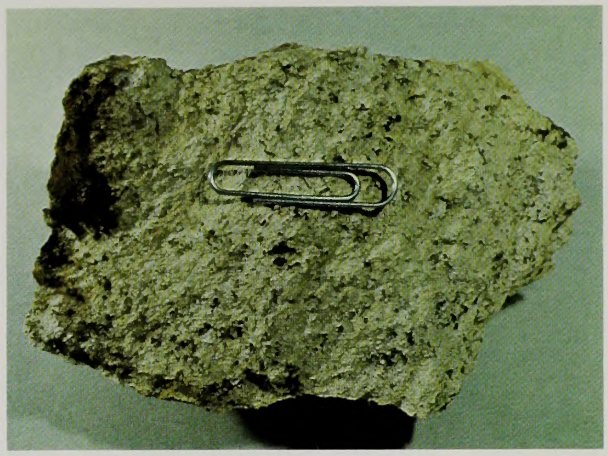


Figure 13. Tuff composed of smaller particles than breccia.

4. Granitoid Rocks

Granite is a light-colored, coarse-grained rock (Figure 14) which is formed when molten material is intruded into surrounding rock; the rate of cooling is very slow and mineral crystals grow large. Variations in the chemical composition of this intrusive material create a wide spectrum of *granitoid* (or granite-like) rocks (Figure 15). Weathered granitoid rock tends to crumble easily into a coarse "sand" as the weathering process degrades the grain-to-grain contact that holds the large crystals together. Granitoid rocks are found in the Klamath Mountains and in isolated spots in the Cascades (Figure 2).

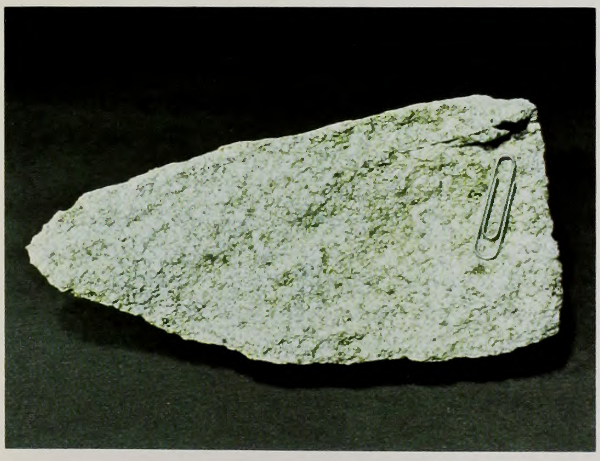


Figure 14. Granite rock, light colored and coarse grained.

Figure 15. Granitoid or "granite-like" rocks

5. Metamorphic Rocks

Metamorphic rocks are formed from any existing rock by changes caused by heat, pressure, and/or chemical alteration. Either the pressure of extremely thick layers of sedimentary rock or the heat from large bodies of intrusive igneous rocks can alter the texture of the surrounding rock with an accompanying rearrangement of elements into different minerals. Ground water can also carry chemicals into or away from the area where metamorphism is taking place. Limestone may be metamorphosed into marble; sandstone into quartzite; and mudstone into *shale* and then into *slate*. Further metamorphism of slate produces *phyllite*, a rock whose color ranges from light-green through gray to black. Freshly broken surfaces of phyllite show a silky sheen. *Schist* is one of the most abundant rocks produced by regional metamorphism. There are many varieties of schist produced from igneous, sedimentary, or metamorphic rocks. Schists are dominated by clearly visible flakes of platy minerals such as mica, talc, or hematite (Figures 16 and 17). Metamorphism of other materials, commonly granitoid rocks, may produce *gneiss* (pronounced "nice"), which is a relatively coarse-grained rock with parallel layers of light and dark minerals (Figure 18).

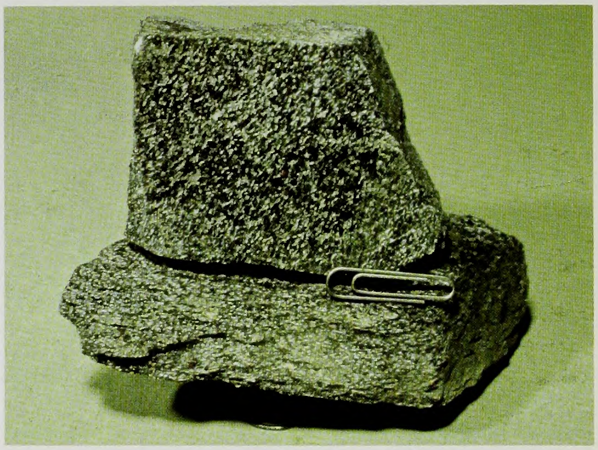


Figure 17. Schist from Evans Creek near Medford, Oregon.

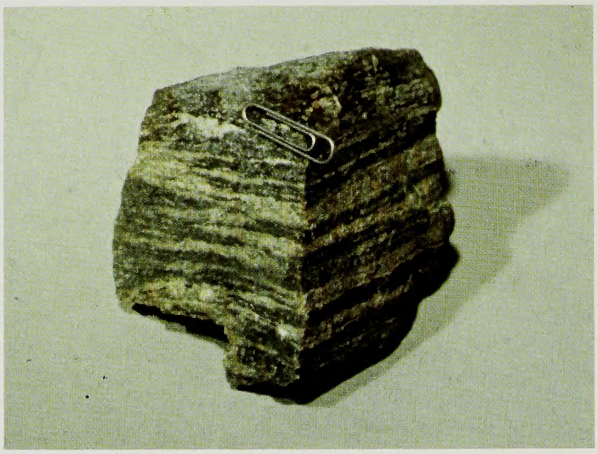


Figure 18. Gneiss has relatively thick bands of colored material.

Figure 16. Colebrook schist showing the platy appearance.

The only reason we see the results of metamorphism is that geologic uplift and erosion exposes the cores of mountains that contain metamorphic rock. When the metamorphic rocks are exposed to the atmosphere they are in a different environment than when formed, and many of the minerals are unstable and break down. The Klamath Mountains have extensive areas of metamorphic rocks of various types.

The metamorphic rocks of southwestern Oregon and northern California can be roughly subdivided into two groups: the Applegate group and the Galice group. The Applegate group is composed mostly of metamorphosed volcanic rocks (often abbreviated as *metavolcanic*) including green to greenish-gray breccia, tuffs, and intrusions of basaltic or andesitic rocks. This group also includes some metamorphosed sedimentary rocks (often abbreviated as *metasedimentary*) such as dark pebbly conglomerate and some marble and quartzite. Rocks of the Applegate group occupy the easternmost portion of the Klamath Mountains in Oregon and most of the southern portion of this province in California (see Figures 1 and 2). The Galice group consists of partly metamorphosed sedimentary rocks from the Dothan, Galice, and Rogue Formations in Oregon, together with metavolcanics. The Dothan Formation extends into California from the southwestern portion of the Galice group in Oregon as shown in Figure 2. In California the Dothan Formation is known as the Franciscan Formation (Dott, 1971) and covers an extensive area along the western and southern edges of the Klamath Mountains and south along the coast to San Francisco Bay (USGS Map I-512).

6. Ultramafic Rocks and Serpentine

There is a group of rocks composed essentially of magnesium-iron-silicate minerals that is designated as *ultramafic* rocks on geologic maps. These rocks were intruded into surrounding igneous or sedimentary rock as a relatively cold mass that caused intensive shearing and crushing, not only of the surrounding rock but of the intruding ultramafic rock as well. As the ultramafic rock was intruded it gathered water from the adjoining rock and absorbed it chemically to produce *serpentine*, especially along the sheared contacts. This process is called *serpentinization.* An examination of highly sheared fault zones within a broad area of ultramafic material may show individual rocks with a thin layer of light-colored serpentine on one or more surfaces, indicating serpentinization of this zone. Serpentinization of ultramafic rocks is so frequent that the geologic map of Figure 2 shows all ultramafic rock as well as veins and masses of pure serpentine grouped into one map unit designated as "serpentine."

Ultramafic rocks are usually dark-colored and fine-grained. Some of the tougher, less sheared ultramafic material may be crushed and used as surfacing in road building. The badly sheared material frequently breaks into thin shards with sharp edges that forms slopes that slide readily.

The color of serpentine ranges from yellow-green, bluish-green, or olive-green to almost black (Figure 19). Serpentine occurs within the ultramafic rocks as described above, but also as intrusions within non-ultramafic rocks. These intrusions may be in the form of thin veins or extensive masses. The thin belts or streaks of serpentine are frequently oriented southwest to northeast in the northern Klamath Mountains and along the southwest coast of Oregon. Farther south in California, these belts of serpentine are oriented northwest to southeast.

Figure 19. Serpentine may be black, bluish, or yellow-green.

GEOLOGIC FEATURES THAT AFFECT SLOPE STABILITY

There are certain geologic features that have a profound effect upon slope stability and can consequently affect road construction in an area. Many of these geologic features can be observed in the field and may also be identified on topographic maps and aerial photographs. In some cases the presence of these geologic features may be deduced by comparing geologic and topographic maps. This section will attempt to illustrate those geologic features that have a significant effect upon slope stability and the techniques that may be used to identify them.

1. Faults

The geologic uplift that accompanies mountain building is quite evident in the Coast Range and Klamath Mountains. Stresses built up in layers of rock by the warping that accompanies uplift is usually relieved by fracturing. These fractures may extend for great distances both laterally and vertically and are known as faults. Often the material on one side of the fault is displaced vertically relative to the other side and sometimes igneous material or serpentine may be intruded into faults. The simplified geologic map of Figure 2 is too small to show individual faults but the orientation of the major veins of serpentine in the Klamath Mountains indicate a pronounced southwest to northeast trend for major fault zones in Oregon and a northwest to southeast trend in California. Larger scale geologic maps also show this trend for major and minor faults over much of the Coast Range. Faults are the focal point for stress relief and for intrusions of igneous rock and serpentine; therefore, fault zones usually contain rock that is fractured, crushed, or partly metamorphosed. It is extremely important to recognize that fault zones are zones of geologic weakness and, as such, are critical in road location. Faults often leave topographic clues to their location and an effort should be made to identify any faults in the vicinity of a proposed road location.

An example of matching a geologic map to a topographic map is shown in Figure 20. An enlarged Oregon township (R. 8 W., T. 13 S.) from a geologic map (Wells and Peck) is shown in the upper corner of a portion of the USGS Alsea Quadrangle topographic map (15 minute series). The faults, shown as heavy black lines on the geologic map, are outlined with heavy black lines in the topographic map. The location of these fault zones is established by looking for:

A. Saddles, or low sections in ridges, which are aligned in the same general direction from one drainage to another.

B. Streams that appear to deviate from the general direction of nearby streams.

Note that the proposed locations of the fault zones on the topographic map follow saddles and drainages in reasonably straight lines.

A rectangular area that includes a portion of the fault zone is outlined on the topographic map in Section 27 of Figure 20, and a stereogram developed from aerial photography of this area is shown in Figure 21. An examination of the stereogram through a stereoscope will show that there is a large slope failure on the fault zone. This is an old failure as indicated by the lack of exposed soil and by the fact that mature timber was harvested from this area. An examination of this area on the ground showed that igneous rock had intruded into the fault, that the slopes at A and B are actively moving, and that the slope at C has slid into the stream since these photographs were taken.

Aerial photographs should be carefully examined for possible fault zones when neither geologic maps nor topographic maps offer any clues. Figure 22 is a stereogram of a portion of Cow Creek, south-southwest of Roseburg, Oregon. A possible fault zone is indicated on the stereogram; note that it passes through several saddles and begins and ends in Cow Creek. Also note the large, old slide at A and the newer slide at B. These photographs were taken in 1965; since then a highway has been constructed along Cow Creek on the bank across from the railroad. The slide at B has enlarged, has been actively moving, and has buckled the highway at the base of the slide. Further evidence of a fault zone through this area is shown in the photograph of a rock from the slide at B (Figure 23). The important feature is the slick, shiny surface caused by the intense heat developed by friction on sliding surfaces within the fault zone.

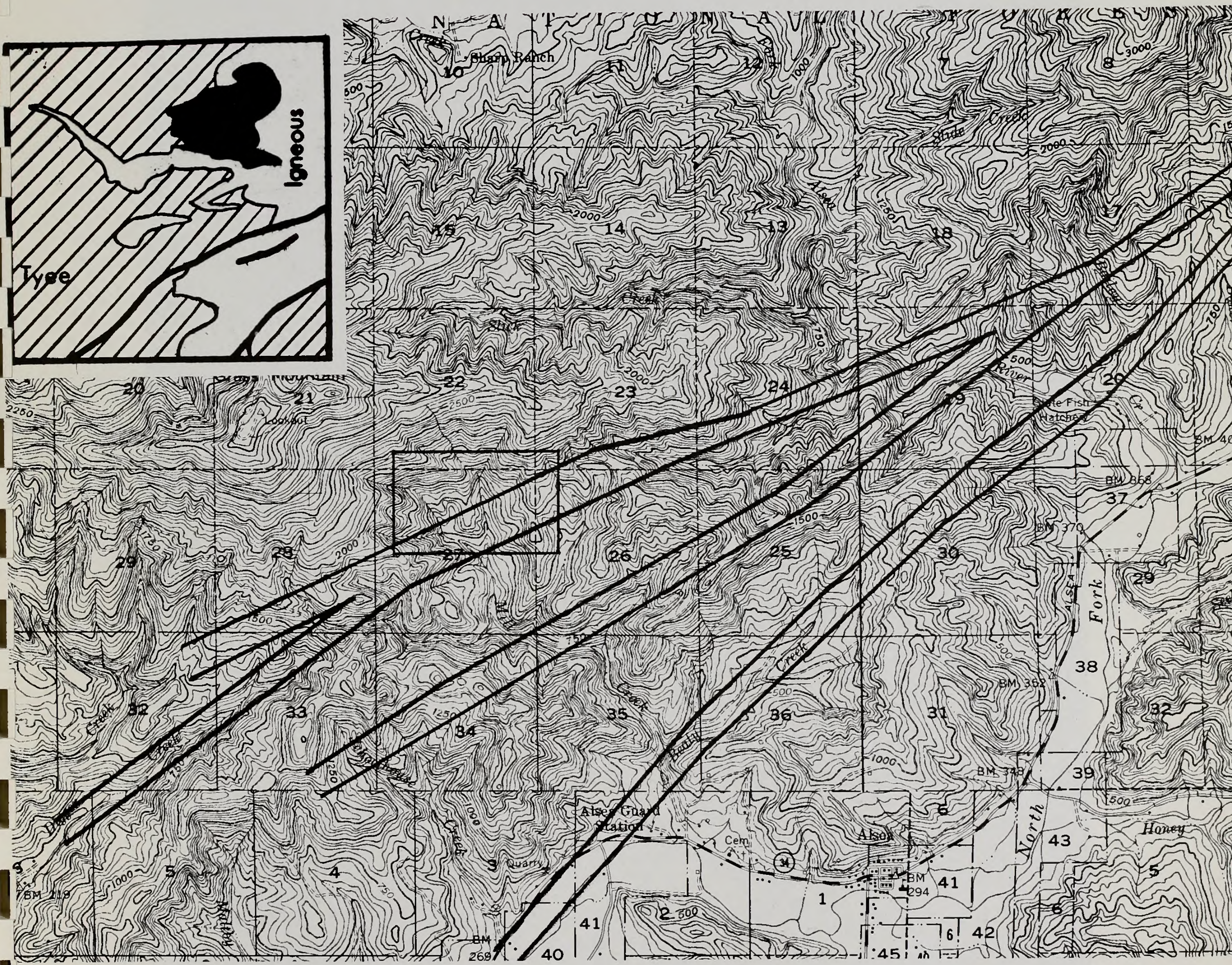


Figure 20. Suspected fault zones are indicated by the alignment of saddles in ridges and by the direction of stream channels. Geologic map of R. 8 W., T. 13 S., is found in upper left corner. Major faults are shown as heavy dark lines on geologic map.

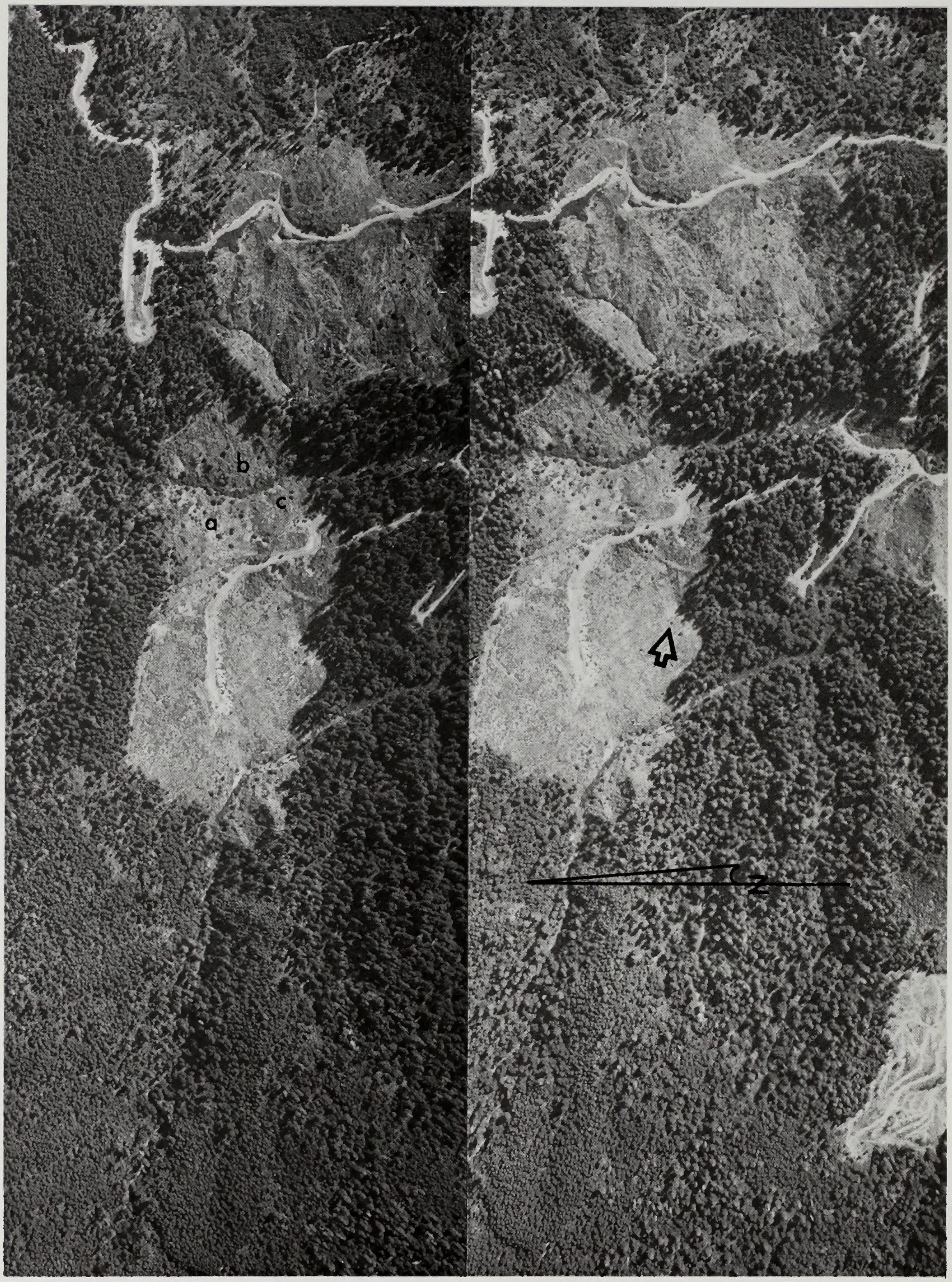


Figure 21. Stereogram of fault zone outlined in Section 27 of Figure 20. Large slump in center of photo; actively failing slopes at a and b. Arrow points to a dry sag pond below the slump.

Figure 22. Stereogram of a possible fault zone. The location of the fault is indicated by the dashed line through the low saddle between the large, older slump at A and the newer slope failure at B.

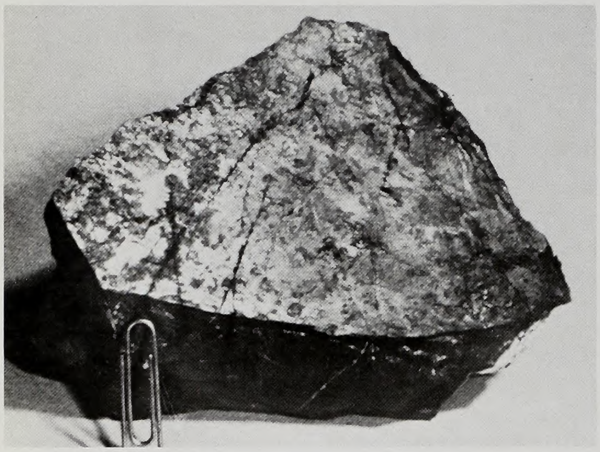


Figure 23. A rock from the rockslide at point B on Figure 22 Note the shiny surface.

Field personnel should be alert for on-the-ground evidence of faulting when neither geologic maps nor topographic maps provide definite clues to the location of faults. Figure 24 is a stereogram of a suspected fault zone. Topographic maps of this area do not show any definite indication of a fault zone. Timber has been harvested from several of the small drainages and fractured and uptilted rock may be seen at Point A in Figure 24. Figure 25 shows a photo of Point A taken from the spur road at B. This direct evidence of faulting probably could not have been seen on aerial photos before timber harvest, but if field personnel observe such evidence during road reconnaissance then an intensive investigation should be started. In this case, the faulting at A together with the topographic clues at C and D indicate a fault zone that extends through C and D.

The purpose of this discussion of faults is to emphasize that geologic maps, topographic maps, information from aerial photos, and on-the-ground clues should all be used to help locate fault zones early in the road location process because faults will increase the probability of slope and road failures.

Figure 25. Photograph of the fault zone shown at point A on Figure 24.

Figure 24. Fractured and up-tilted rocks are shown at point A. A ground photo of this feature photographed from point B is shown in Figure 25. Changes in the slope of the ground at points C and D indicate a possible fault zone through these points.

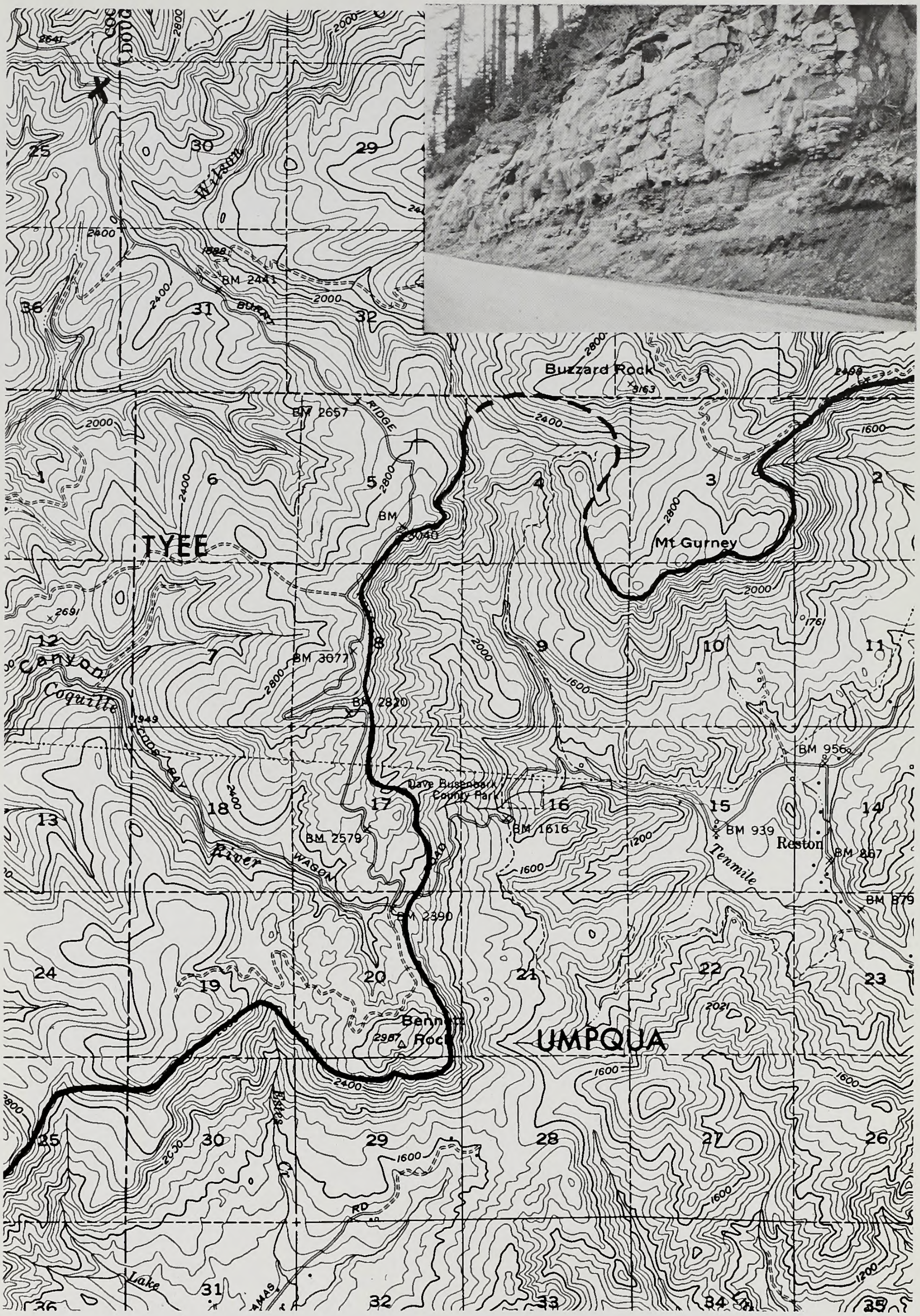


Figure 26. Topographic map of the Camas Valley Quadrangle. The Tyee sandstone is a more competent formation than the Umpqua formation in this area, and forms cliffs at the junction of these two formations. A photo of the contact between these two formations is shown in the upper right. The location of this photo is shown by the X in the upper left.

2. Strength of the Rock and Resistance to Erosion

A second set of geologic features is concerned with properties of the rock material: the competence, or inherent strength of the rock, and the ability of the rock to resist weathering. The term competence, as defined by engineering geologists, refers to the ability of the rock material to resist compression, tension, etc., without crumbling or fracturing. Competence depends upon the material itself, how badly it is fractured and the thickness of the beds or layers. Thin-bedded material, or thick-bedded rock that is badly fractured, may be relatively incompetent. The ability of a rock to withstand the weathering process is also an important factor in road construction. For example, massive granite can be considered to be more competent than sandstone, but the ease with which some granitoid rocks disintegrate once they have been exposed during construction usually creates more road maintenance problems than does sandstone.

Geologic maps and topographic maps when used together can help locate, relatively accurately, the boundary between geologic materials with different values of competence and resistance to weathering. Figure 26 is a portion of a topographic map of the Camas Valley quadrangle west-southwest of Roseburg, Oregon. Note the change in slope gradient from the Tyee sandstone to the underlaying Umpqua Formation. The boundary of these two formations is indicated by the line. The Tyee Formation is more competent and has a greater resistance to weathering than does the Umpqua Formation. This difference is indicated by the cliff-forming character of the Tyee Formation. The Umpqua Formation weathers rapidly to moderate slopes because of the high percentage of siltstone in the formation in this area. A picture of the contact between the Tyee and darker Umpqua Formation is inserted in the upper corner of Figure 26. The approximate location of this photo is indicated by the X in the upper left corner of the map.

3. Slope of Bedding Planes

There are many locations where sedimentary or metamorphic rocks have been warped or tilted and the bedding planes may be steeply sloped. If a road is planned for such an area, it is important to determine the slope of the bedding planes relative to the ground slope. In areas where bedding planes are approximately parallel to the slope of the sidehill, road excavation may remove enough support to allow large chunks of rock and soil to slide into the road (Figure 27). If preliminary surveys reveal that these conditions exist, then the route may need to be changed to the opposite side of the drainage or ridge where the bedding planes slope into the hillside.

Figure 27. This figure shows how easily soil can slide into a road when the bedding planes of the rock slope toward the road.

SOIL MECHANICS AND SLOPE STABILITY

Slope gradient and ground water are the two factors that have the greatest effect upon slope stability. Generally, the greater the slope gradient and the more ground water present, the less stable will be a given slope regardless of the geologic material or the soil type. It is absolutely essential that engineers, foresters, and technicians engaged in locating, designing, constructing and maintaining roads understand why slope gradient and ground water are so important to slope stability. This section will describe in simple terms how slope and ground water interact with various soils and geologic materials to affect slope stability.

SLOPE GRADIENT

The effects of slope gradient on slope stability can be understood by discussing the stability of a pure, dry sand. Slope stability in sand depends entirely upon frictional resistance to sliding. Frictional resistance to sliding, in turn, depends upon: 1) the slope gradient which affects that portion of the weight of an object that rests on the surface, and 2) the coefficient of friction. The fraction of the weight of an object that rests on a surface is known as the *normal force* because it acts normal to, or perpendicular to, the surface. The normal force changes as the slope of the surface changes. The upper curve in Figure 28 shows how the gradient of a surface changes the normal force (N) of a 100# block resting on the surface. As expected, when the slope gradient is zero the entire weight of the 100# block rests on the surface and the normal force = 100#. When the surface is vertical there is no weight on the surface and the normal force is zero. The coefficient of friction converts the normal force (N) to frictional resistance to sliding (F). An

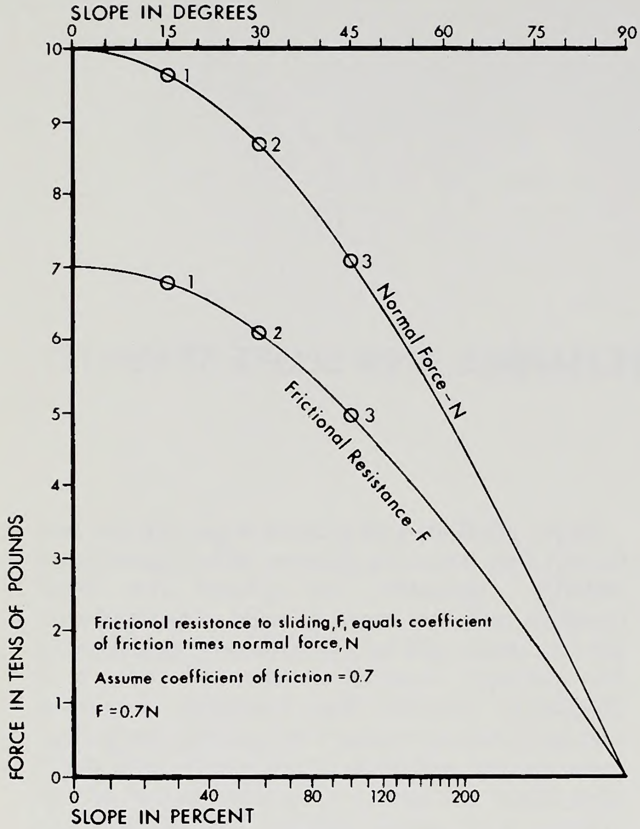


Figure 28. The upper curve shows that the portion of a 100# weight resting on a surface changes as the slope of the surface changes. The lower curve shows how the frictional resistance to sliding changes with the slope of the surface.

average value for the coefficient of friction for sand is about 0.70. This means that the force required to slide a block of sand along a surface is equal to 0.7 times the normal force. The lower curve in Figure 28 shows how the frictional resistance to sliding changes with slope gradient. The lower curve was developed by multiplying the values of points on the upper curve by 0.7. Therefore, when the slope gradient is zero, the normal force = 100# and 70# of force is required to slide the block along the surface. When the slope gradient is 100 percent, the normal force = 71# (from point 3 on the upper curve), and 50# (or 71# x .7) is required to slide the block along the surface (from point 3 on the lower curve).

That portion of the weight which acts downslope, or parallel to the surface, provides some of the force to overcome frictional resistance to sliding. The downslope force, sometimes known as the *driving force*, also depends upon the slope gradient and increases as the gradient increases, as shown in Figure 29. Obviously when the slope gradient becomes steep enough the driving force will exceed the frictional resistance to sliding and the block will begin to move.

Figure 30 shows the curve for frictional resistance to sliding (from Figure 28) superimposed on the curve of the driving force (from Figure 29). These two curves intersect at 70 percent (35°) slope gradient. In this example, this means that for slope gradients less than 70 percent, the frictional resistance to sliding is greater than the downslope component of the weight of the block and the block will remain in place on the surface. For slope gradients greater than 70 percent, the block will slide because the driving force is greater than the frictional resistance to sliding. Appendix A gives a more detailed discussion of frictional resistance to sliding. This discussion has been confined to the case of a pure, dry sand, a case which is seldom found in soils, but the principles of the effects of slope gradient and frictional resistance to sliding apply to any dry soil.

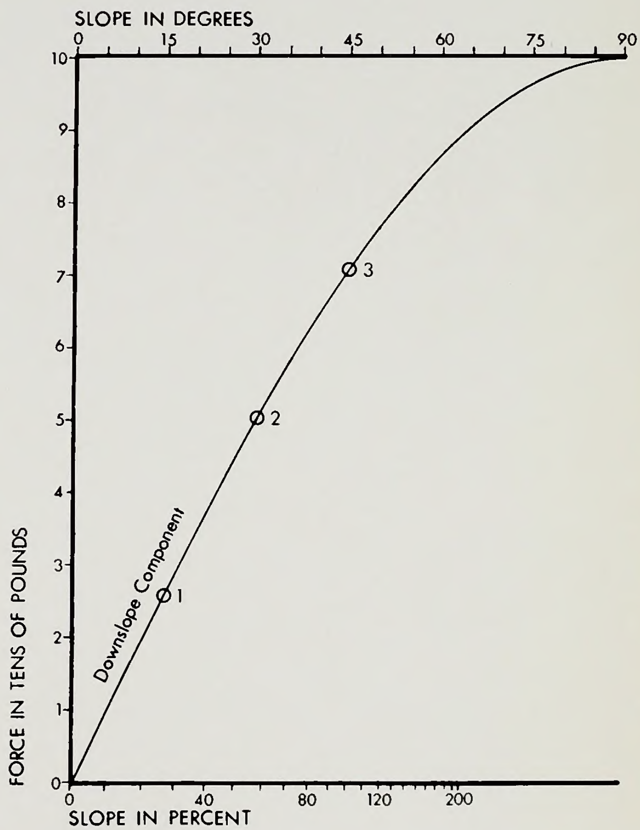


Figure 29. This curve shows that the downslope component of a 100# weight increases as the slope of the surface increases.

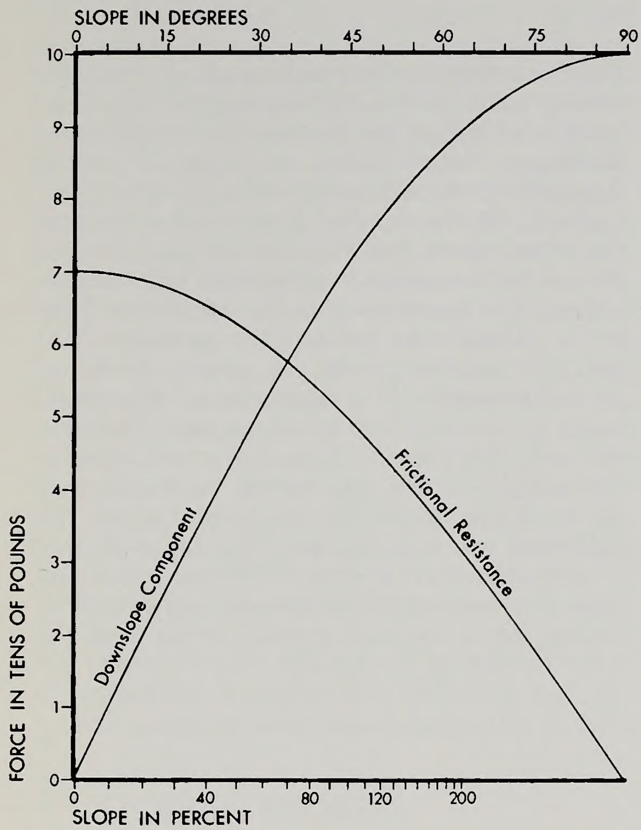


Figure 30. This figure shows how the frictional resistance to sliding compares with the downslope component of weight and how these two forces determine the stability of an object. At slopes less than 70% (or 35°), the object will not slide; at slopes greater than 70%, the object will slide.

A block of uniform soil fails, or slides, by *shearing*. That is, one portion of the block moves past another portion in a parallel direction. The surface along which this shearing action takes place is called the *shear plane*, or the *plane of failure*. The resistance to shearing is often referred to as *shear strength*. Pure sand develops shear strength by frictional resistance to sliding, but pure clay is a sticky substance which develops shear strength because the individual particles are cohesive, that is, they stick to each other. The presence of clay in soils increases the shear strength of the soil over that of a pure sand because of the cohesive nature of the clay (see Appendix B).

A dry clay has considerable shear strength as demonstrated by the great force required to crush a clod with the fingers. However, as a dry clay absorbs water, its shear strength decreases because water films tend to separate the clay particles, and thus reduce its cohesive strength. The structure of the clay particle determines how much water will be absorbed and consequently, how much the shear strength will decrease upon saturation. There are some clays, such as illite and kaolinite, that provide relative stability to soils, even when saturated. However, a saturated montmorillonite clay causes a significant decrease in slope stability. Saturated illite and kaolinite clays have about 44 percent of their total volume occupied by water compared to about 97 percent for a saturated montmorillonite clay. This explains why montmorillonite clay has such a high shrink-swell potential (i.e., large change in volume from wet to dry) and saturated clays of this type have a very low shear strength. Thus, the type of clay in a forest soil has a significant effect upon slope stability (Paeth, et al.).

Granitoid rocks tend to weather to sandy soils as the weathering process destroys the grain-to-grain contact that holds the mineral crystals together. If these soils remain in place sufficiently long, they will eventually develop a significant amount of clay. If erosion removes the weathered material at a rapid rate, the resulting soil will be coarse-textured and will behave as a sand for purposes of slope stability analysis. It can be observed in many locations that soils with a significant clay content that have developed from granitoid material will have greater shear strength and will support steeper cut faces than soil with little clay also from granitoid rocks.

This discussion shows that for two soils developed from the same geologic material, the soil with the higher percentage of illite or kaolinite clay will have greater shear strength than soils with significant amounts of montmorillonite clay. The relative stability among soils depends on a comparison of their shear strength and the downslope component of the weight of the soil.

GROUND WATER

A common observation is that a hillslope or the sideslopes of a drainageway may be perfectly stable during the summer but may slide after the winter rains begin. The reason for this seasonal change in stability is due mainly to the change in the amount of water in the pores of the soil. The effect of ground water on slope stability can best be understood by again considering the block of pure sand. Frictional resistance to sliding in dry sand is developed as the product of the coefficient of friction and the normal force acting on the surface of the failure plane. A closeup view of this situation would show that the individual sand grains are interlocked, or jammed together, by the weight of the sand. The greater the force that causes this interlocking of sand grains, the greater will be the ability to resist the shear force that is caused by the downslope component of the soil weight. As ground water rises in the sand, the water reduces the interlocking force because of the buoyant force exerted on each sand grain as it becomes submerged. This buoyant force is easily demonstrated by first weighing an object with a spring scale, then submerging the object in water and noting the decrease in the weight of the object. The difference in weight is caused by the uplift force of the water acting on the object.

The uplift force of the ground water reduces the interlocking force on the soil particles and this, in turn, reduces the frictional resistance to sliding. The *interlocking force* is the same thing as the *normal force* that was discussed at the first part of this section; these two terms will be used interchangeably. The uplift force of ground water is equal to 62.4 pounds per foot of water in the soil. The *effective normal force* is equal to the weight of the soil resting on the surface minus the uplift force of the ground water. The following example illustrates the calculation of the effective normal force. If 100 pounds of sand rests on a horizontal surface and contains three inches (or ¼ foot) of ground water then the effective normal force is 100 - 62.4 (¼) = 100 - 15.6 or 84.4 pounds. The frictional resistance to sliding with this ground water condition is 84.4 x

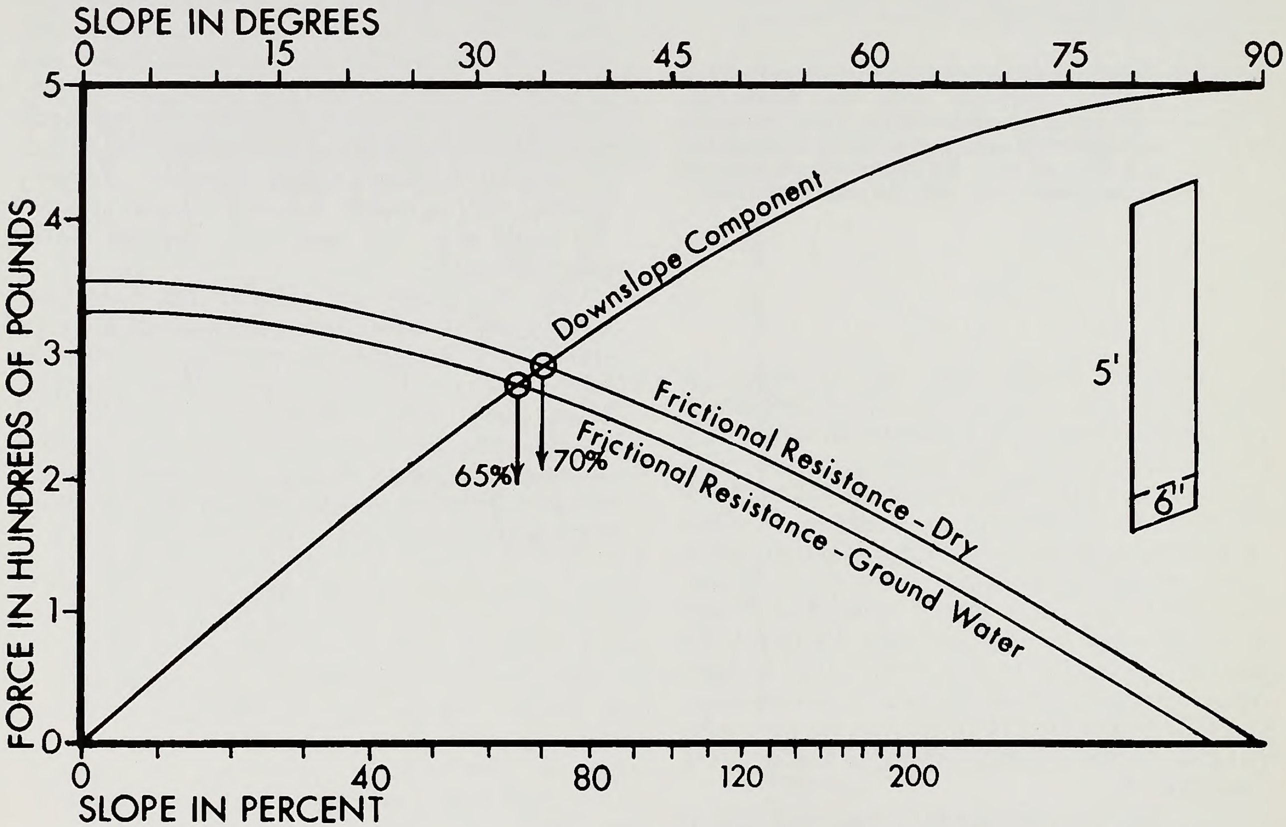


Figure 31. Five feet of soil with 6 inches of ground water will slide when the slope gradient exceeds 65%.

.7 or 59.1 pounds. As a comparison, for 100 pounds of dry sand on a horizontal surface the frictional resistance to sliding is 100 x .7 or 70 pounds. This example shows how ground water can reduce the frictional resistance to sliding.

The following examples further emphasize how the presence of ground water can decrease slope stability. First, a layer of dry sand five feet thick is assumed to weigh 100 pounds per foot of depth. The downslope component of the dry weight and the frictional resistance to sliding for dry sand was calculated for various slope gradients as in Figures 28 and 29. Next, six inches of ground water is assumed to be present and the frictional resistance to sliding is recalculated taking into account the uplift force of the ground water. The results of these calculations are shown in Figure 31. Note that in a dry condition, sliding will occur when the slope gradient exceeds 70 percent. With six inches of ground water, the soil will slide when the slope gradient exceeds 65 percent. For a comparison, assume a dry sand layer only two feet thick that weighs 100 pounds per foot of depth. Again, assume six inches of ground water and recalculate the downslope component of the soil weight and the frictional resistance to sliding with, and without, the ground water. These results are shown in Figure 32. Note that with six inches of ground water, this thin layer of soil will slide when the slope gradient exceeds 58 percent.

These examples demonstrate that the thinner soil mantle has a greater potential for sliding under the same ground water conditions than a thicker soil mantle. The six inches of ground water is a greater proportion of the total soil thickness for the two-foot soil than for the five-foot soil, and the ratio of uplift force to the frictional resistance to sliding is greater for the two-foot soil. A pure sand was used in these examples for the sake of simplicity, but the principles still apply to soils that contain varying amounts of silt and clay together with sand.

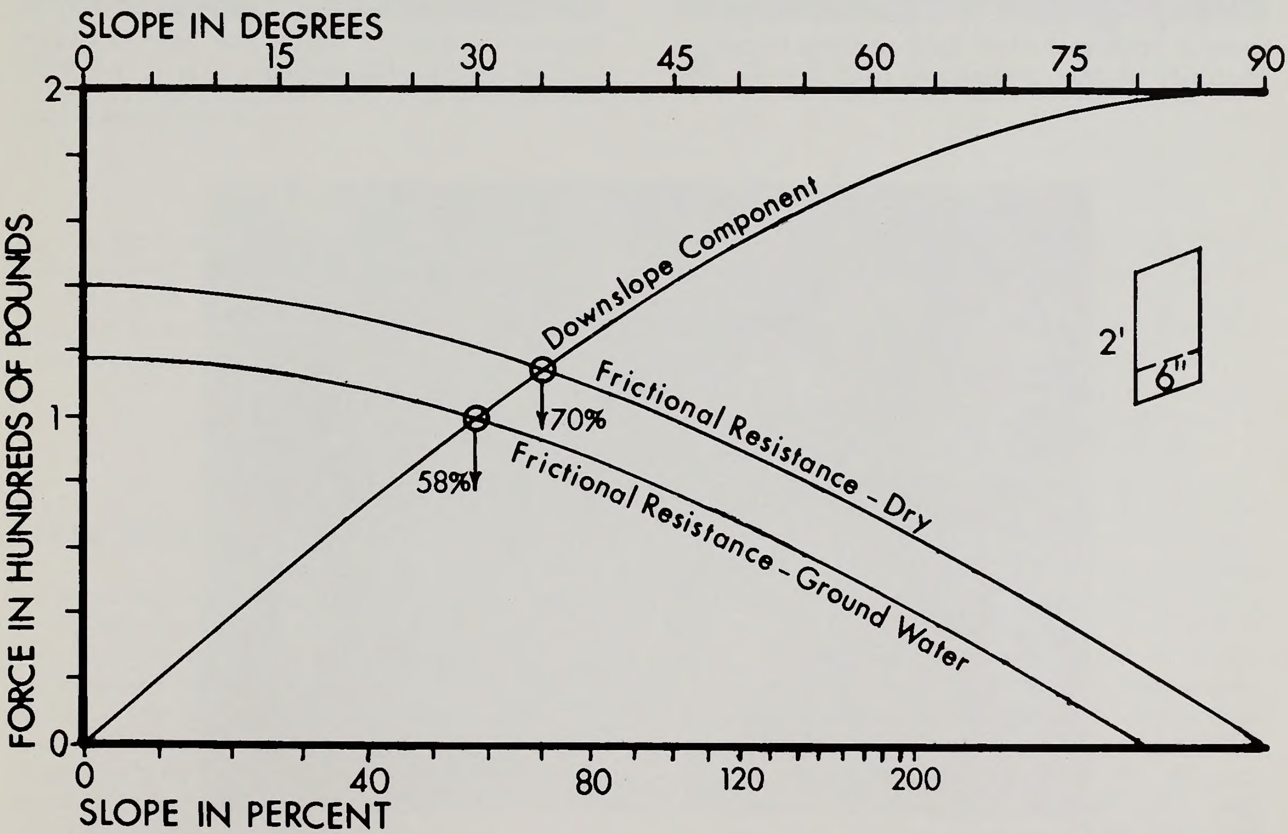


Figure 32. Two feet of soil with 6 inches of ground water will slide when the slope gradient exceeds 58%.

SEEPAGE FORCE

There is still another way that ground water contributes to slope instability, and that is the *seepage force* of ground water as it moves downslope. The seepage force is the drag force that moving water exerts on each individual soil particle in its path. The seepage force, therefore, contributes to the driving force that tends to move masses of soil downslope. The concept of the seepage force may be visualized by noting how easily portions of a coarse-textured soil may be dislodged from a road cut bank when the soil is conducting a relatively high volume of ground water.

TREE ROOTS

The effect of tree root strength on slope stability is not fully understood and is the subject of studies by government agencies and various universities. Preliminary results of these studies indicate that under certain conditions living tree roots help maintain slope stability. Reports by the Forest Service from southeast Alaska indicate that the number of landslides from cut-over areas increases within 3 to 5 years after logging. This increase is attributed to a reduction in soil shear strength caused by the decay of tree roots following logging. The presence of living tree roots to anchor shallow soils on steep slopes to fractures in the underlying rock (Figure 33) appears to be particularly important in small drainages where winter storms can cause the ground water level to rise sharply (Swanston, 1969, 1970). Similar correlations between the number of landslides following timber cutting and steep slopes with heavy winter rainfall have been noted in other parts of the United States and other countries (Gray, 1973).

Figures 34 and 35 show typical slope failures that frequently originate within clearcut areas in shallow depressions on steep side slopes. Figure 36 is a stereogram that shows a number of failures on clearcut slopes that have not been roaded. All studies indicate that if the soils,

Figure 33. This illustrates the manner in which tree roots act to hold shallow soil to fractured bedrock.

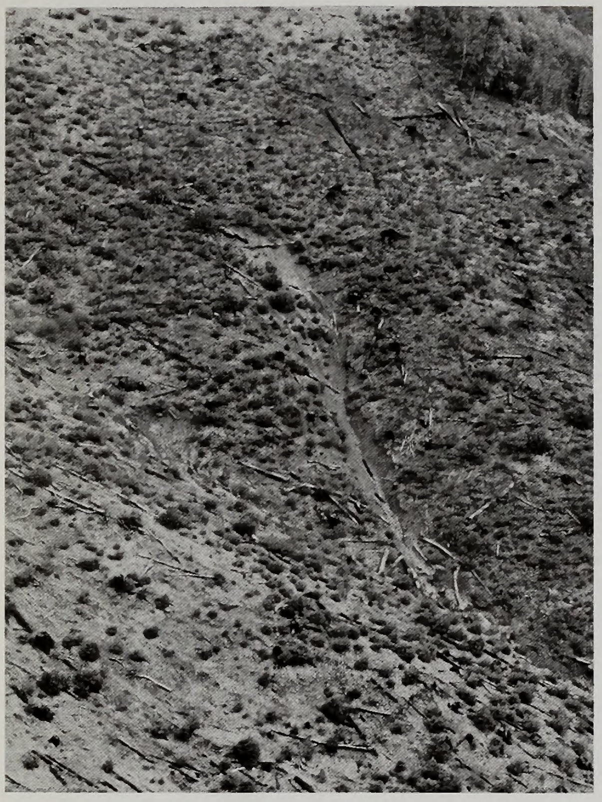


Figure 34

Figure 35

These photographs show shallow landslides that originate on clearcut areas. They often occur in small drainages where ground water may accumulate.

topography, and vegetation show that an area with mature timber presently has marginal slope stability, then serious consideration should be given to the effect that removal of this timber will have on future slope stability.

TYPES OF SLOPE FAILURES

Slope failures include all mass soil movements on 1) man-made slopes such as road cuts and fills, or 2) natural slopes in clearcut areas or undisturbed forest. A classification of slope failure is useful because it provides a common terminology and it offers clues to the type of slope stability problem that is likely to be encountered for a given soil, geologic material, and topography. It has been shown that types of slope and road failures are remarkably consistent with soils, geologic material, and topography. For example, fast-moving debris avalanches or slides develop in shallow, coarse-textured soils on steep hillsides, and large, rotational slumps occur in deep, saturated soils on gentle to moderate slopes. A survey of road failures over an 18-month period in the Eugene District of the Bureau of Land Management in Oregon showed that 71% of the road failures in bedded sediments occurred on slopes over 60%, as compared to only 34% of the road failures in volcanic materials. This indicates that the coarse-textured soils that develop from sedimentary material on steep slopes in this area tend to fail as debris slides. The volcanic materials tend to develop finer textured soils that fail as slumps and earthflows on the more moderate slopes. The same survey showed that 66% of the road failures in bedded sediments were cut slope failures as compared to 91% in volcanic materials. The following sections will help explain why certain types of slope and road failures are correlated with soils, geologic material, and topography.

Figure 36. This stereogram shows slope failures that have originated on clearcut areas. The failures outlined in black are not associated with roads.

1. Rockfalls and Rockslides

Rockfalls and rockslides most commonly originate in bedded sediments such as massive sandstone whose beds are undercut by stream erosion or road excavation. Stability is maintained by the competence of the rock and by the frictional resistance to sliding along the bedding planes; these factors are particularly important where the bedding planes dip downslope toward a road or stream. Rockslides occur suddenly, slide with great speed, and sometimes extend entirely across the valley bottom. Slide debris consists of fractured rock and may include some exceptionally large blocks. Rockslides can range in size from the 40,000,000 cubic yard Madison River slide of 1959 in Montana, that killed 28 people, to the 80,000 cubic yard Camp Creek slide in Coos Bay of 1970 (Figure 37). The Cow Creek slide shown at point B in Figure 22 is a rockslide that occurred in the Dothan formation along a fault zone. Figure 38 shows the deposition of fractured rock from this slide and the displacement of the road, both vertically and laterally. This particular slide has a potential for further movement as evidenced by cracks in the slopes parallel to the margin of the slide. Road locations through areas with a potential for rockslides should be examined by specialists who can evaluate the competence of the rock and determine the dip of the bedding planes.

Figure 37. A rockslide in bedded sedimentary material

Figure 38. A photograph of the rockslide illustrated at point B in Figure 22. Note how the shifting rock is lifting the road (arrow) and moving it to the right towards Cow Creek.

2. Debris Avalanches and Debris Flows

These two closely related types of slope failures usually originate on shallow soils relatively low in clay content on slopes over 65 percent (Figures 39 and 40). In southeast Alaska, the Forest Service has found that debris avalanches develop on slopes greater than 65 percent on shallow, gravelly soils and that this type of slope failure is especially frequent on slopes over 75 percent (Swanston, 1969 and 1970). An excellent description of debris avalanches and flows is given in Bailey, parts of which are quoted below:

> "Debris avalanches are the rapid downslope flowage of masses of incoherent soil, rock, and forest debris with varying water content. More specifically, they are shallow landslides resulting from frictional failure along a slip surface essentially parallel to the topographic surface, formed where the accumulated stresses exceed the resistance to shear. The detached soil mantle slides downslope above an impermeable boundary within the loose debris or at the unweathered bedrock surface and forms a disarranged deposit at the base. Downslope, a debris avalanche frequently becomes a debris flow because of substantial increases in water content . . . They are caused most frequently when a sudden influx of water reduces the shear strength of earth material on a steep slope, and they typically accompany heavy rainfall."

There are two situations where these types of slope failure occur in those areas with shallow soil, steep slopes and heavy winter rainfall.

The first situation is an area where stream development and geologic erosion have formed high ridges with long slopes, and steep, V-

Figure 39.

Figure 40.

These photographs illustrate debris avalanches that are common in shallow soils on steep slopes.

shaped drainages usually in bedded sedimentary rock. The gradient of many of these streams increases quite sharply from the main stream to the ridge, and erosion has created headwalls in the upper reaches. The bowl-shaped headwall region is often the junction for two or more ephemeral stream channels which begin at the ridgetop. This leads to a quick rise in ground water levels during winter rains. Past debris avalanches may have scoured round-bottom chutes, or troughs, into the relatively hard bedrock (Figure 41). The headwall region may be covered with only a shallow soil mantle of precarious stability, and it may show exposed bedrock often dark with ground water seepage (Figure 42).

The second situation with a high potential for debris avalanches and flows is caused by excavated material sidecast onto slopes greater than 65%. The sidecast material next to the slope maintains stability by frictional resistance to sliding and by mechanical support from brush and stumps. As more material is sidecast, the brush and stumps are buried and stability is maintained solely by frictional resistance to sliding. Since there is very little bonding of this material to the underlying rock, the entire slope is said to be *overloaded*. It is quite common for new road fills on steep overloaded slopes to fail as debris avalanches and flows as soon as the winter rains saturate this loose unconsolidated material. Under these circumstances the road fill together with a portion of the underlying natural slope may form the debris avalanche (Figure 43). See Appendix C for a more detailed discussion of slope stability analysis of slopes prone to fail as debris avalanches.

Debris avalanches and debris flows occur suddenly, often with little advance warning. There is practically nothing that can be done to stabilize a slope which shows signs of an impending debris avalanche. The best possible technique that can be used to prevent these types of slope failures is to avoid those areas with a high potential for debris avalanches and to avoid overloading steep slopes with excessive sidecast. Foresters and engineers should learn the vegetative and soil indicators of this type of unstable terrain, especially for those areas with high seasonal ground water levels.

Figure 41. Frequent debris avalanches may create round-bottomed chutes in the bedrock.

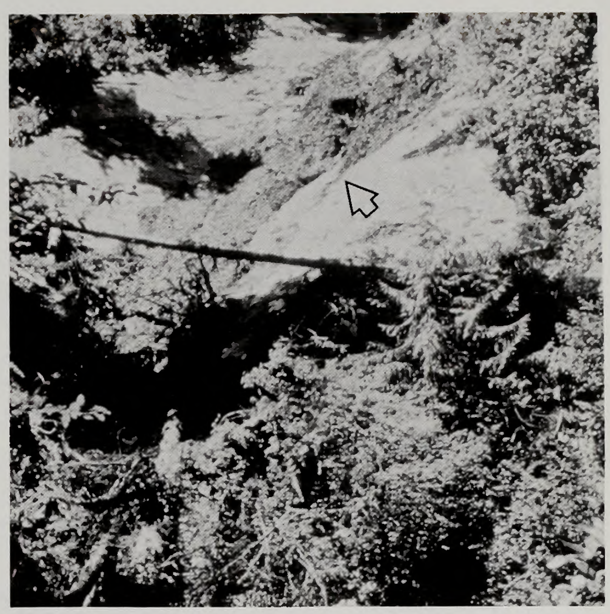


Figure 42. The arrow indicates loose, unstable, material dark with ground water.

If unstable terrain must be crossed by roads, then radical changes in road grade and road width may be required to minimize site disturbance. End-hauling of excavated material may be necessary to keep overloading of unstable slopes to an absolute minimum. The location of safe disposal sites for end-hauled material may be a serious problem in steep terrain with sharp ridges. Selection of these sites will require just as much attention to the principles of slope stability as the location and construction of the remainder of the road. Some specific techniques for identification of unstable terrain, for slope stabilization and road construction will be given later in this publication.

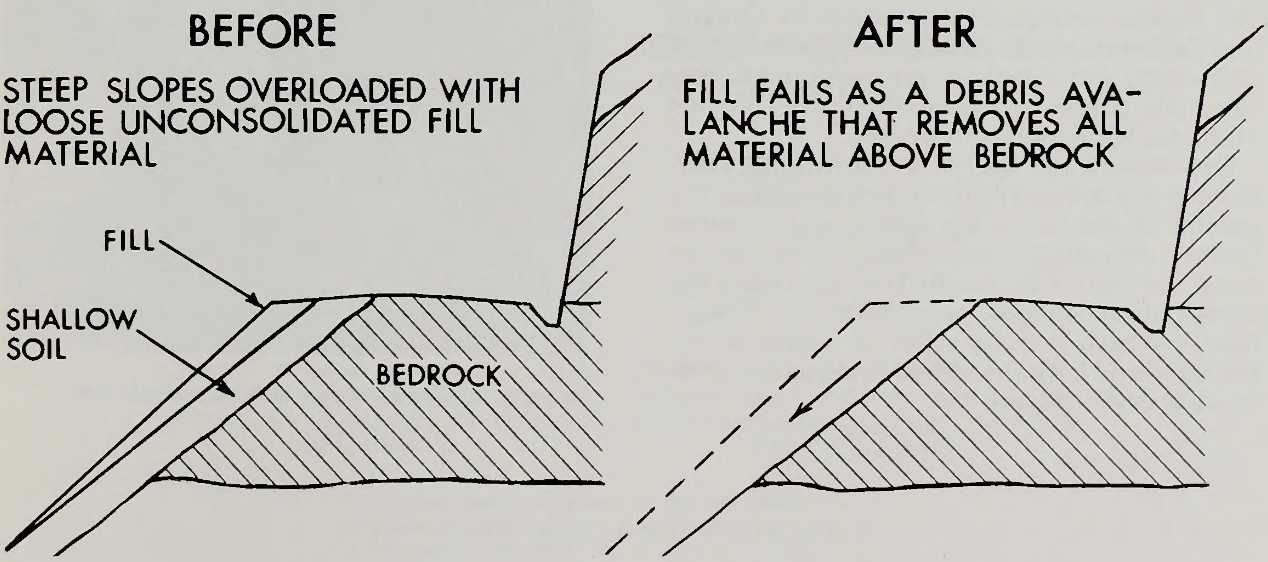


Figure 43. The relationship between overloaded slopes and debris avalanches is shown in this schematic diagram.

3. Slumps and Earthflows

Slumps and earthflows usually occur in deep, moderately fine or fine-textured soils that contain a significant amount of silt and/or clay. In this case, shear strength is a combination of cohesive shear strength and frictional resistance to sliding. As noted earlier, ground water not only reduces frictional resistance to shear, but it also sharply reduces cohesive shear strength. Slumps are slope failures where one or more blocks of soil have failed on a hemispherical, or bowl-shaped, slip surface and they may show varying amounts of backward rotation into the hill in addition to downslope movement (Figure 44). The lower part of a typical slump is displaced upward and outward like a bulbous toe. The rotation of the slump block usually leaves a depression at the base of the main scarp. If this depression fills with water during the rainy season, then this feature is known as a *sag pond*. Figure 21 shows a classic example of a slump; note the sag pond (dry at the time of the photo) and the bulging toe. Another feature of large slumps is the *"hummocky"* terrain, composed of many depressions and uneven ground that is the result of continued earthflow after the original slump. Some areas that are underlain by particularly incompetent material, deeply weathered and subject to heavy winter rainfall, show a characteristically hummocky appearance over the entire landscape. This jumbled and rumpled appearance of the land is known as *mélangé* terrain (pronounced may-lahnj). Examples of this are seen in large areas of the Otter Point Formation in southwestern Oregon, the Dothan-Franciscan Formation in Oregon and California, the Galice Formation in northern California, and deeply weathered sedimentary materials, particularly siltstone, in the Tyee and Nestucca Formations in Oregon.

Depressions and sag ponds allow winter rains to enter the ground water reservoir, reduce the stability of the toe of the slump, and promote further downslope movement of the entire mass. The mature timber that usually covers old slumps often contains *"jackstrawed"* or *"crazy"* trees that lean at many different angles within the stand and indicate unstable soils and actively moving slopes (Figure 45).

Figure 44. Backward rotation of a slump block as it starts its downslope movement to the right.

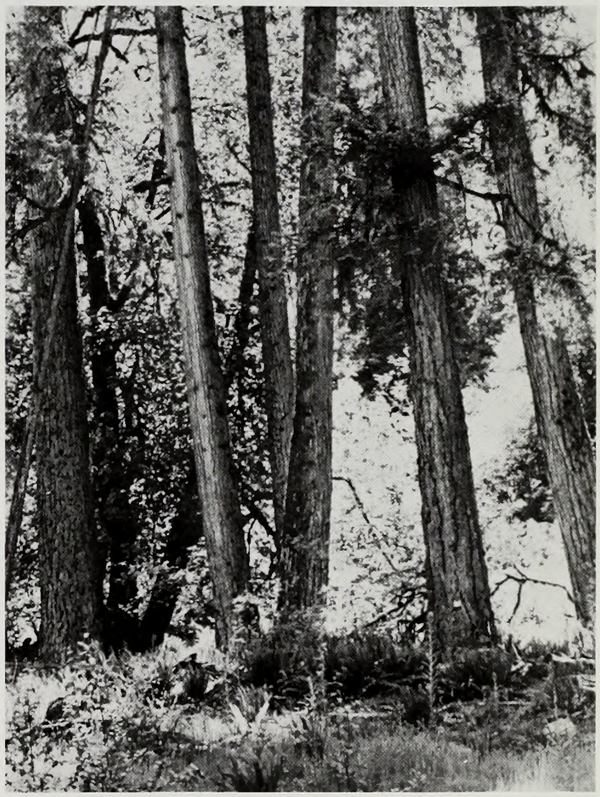


Figure 45. Jackstrawed trees indicate unstable soil

There are several factors affecting slumps that need to be examined in detail to understand how to prevent or remedy this type of slope failure. The block of soil that is subject to slumping can be considered to be resting on a potential failure surface of hemispherical shape (Figure 46). The block is most stable when its center of gravity is at its lowest position on this failure surface. When the block fails, its center of gravity is shifted to a lower, more stable position as a result of the failure. Added weight, such as a road fill, at the head of a slump shifts the center of gravity of the block to a higher, more unstable position and tends to increase the potential for rotation. Similarly, removing weight from the toe of the slump, as in excavating for a road, also shifts the center of gravity of the block to a higher position on the failure surface. Therefore, loading the head of a slump and/or unloading the toe will increase the potential for further slumping on short slopes. The chance of slumping can be reduced by shifting the center of gravity of a potential slump block to a lower position by following the rule: "Unload the head and load the toe." See Appendix D for a detailed discussion of stability analysis of potential slump blocks.

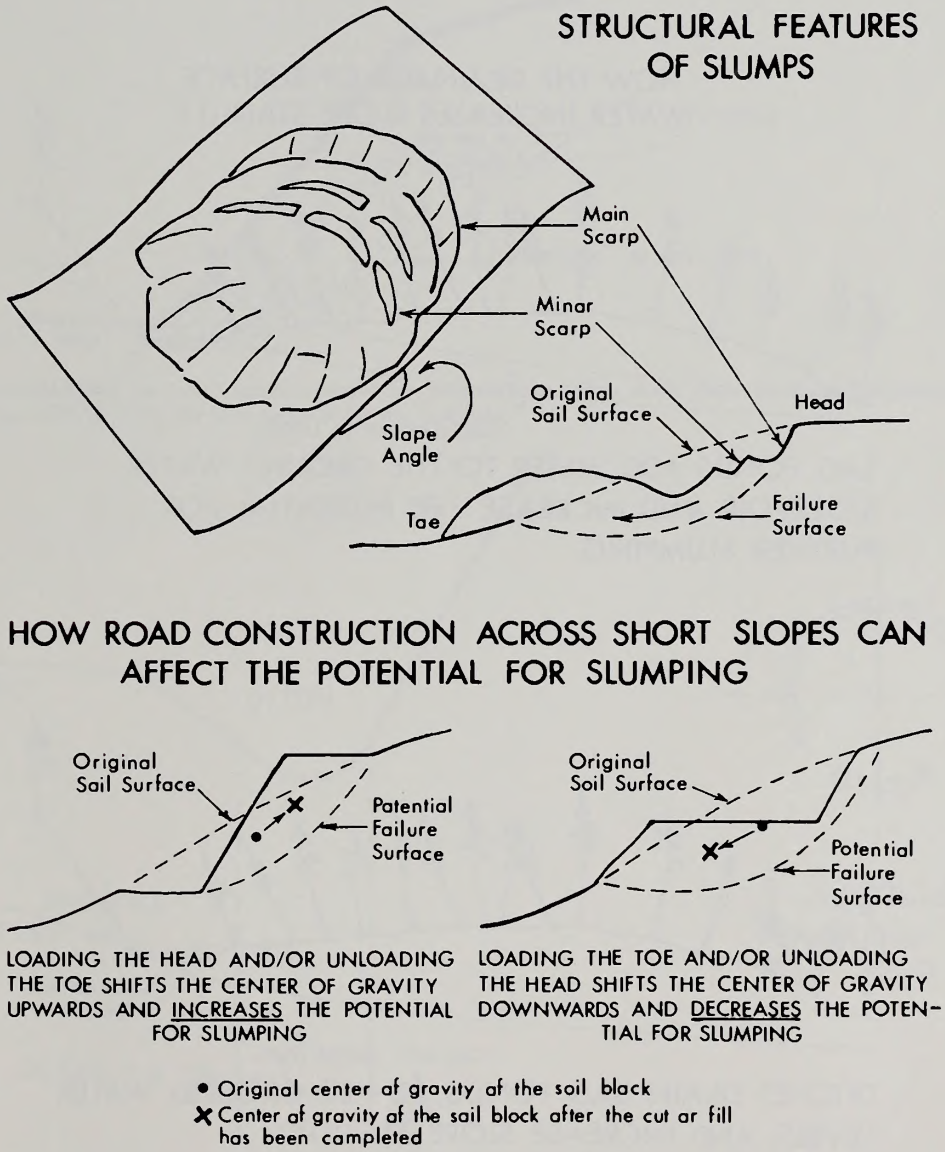


Figure 46. Structural features of slumps and the effect of cutting and filling on the stability of short slopes.

Assuming that it is absolutely necessary to locate a road through terrain with a potential for slumping, there are several techniques that may be considered to prevent slumps and earthflows. Improved surface drainage is one of the cheapest and most effective techniques and one that is often overlooked. Sag ponds and depressions can be connected to the nearest stream channel with ditches excavated by bulldozer or ditching powder. Figure 47 shows the theoretical effect on the ground water reservoir of a surface drainage project. Improved drainage removes surface water quickly, lowers the ground water level, and helps stabilize the slump.

Another technique is to lower the ground water level by means of perforated pipe that is augered into the slope at a slight upward angle. These drains are usually installed in road cutbanks to stabilize areas above an existing road, or below roads to stabilize fills. Installation of perforated pipe is relatively expensive and there is a risk that slight shifts in the slump mass may render the pipe ineffective. In addition, periodic cleaning of these pipes is necessary to prevent blockage of the pipe by algae, soil, or iron deposits.

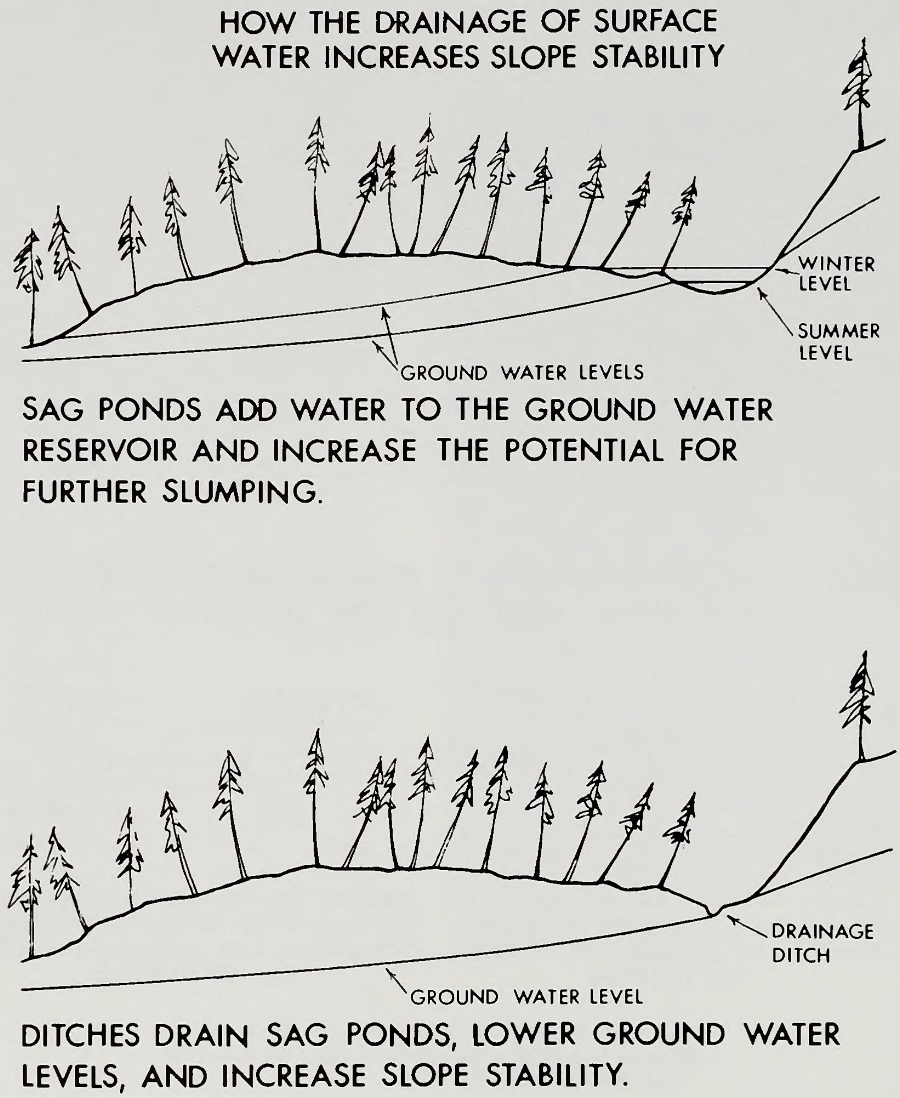


Figure 47. Drainage of surface water can increase slope stability by lowering the ground water level.

A third technique for stabilization of existing slumps and the prevention of potential slumps is to use rock riprap, or buttresses, to provide support for road cuts or fills (Figure 48). Heavy rock riprap replaces the stabilizing weight that is removed by excavation during road construction (refer back to Figure 46). Another feature of riprap is that it is porous and it allows ground water to drain out of the slump material while providing support for the cut slope.

A fourth technique is the installation of an interceptor drain to collect ground water that is moving laterally downslope, under the road, and saturating the road fill. A backhoe can be used to install interceptor drains in the ditch along an existing road. Figure 49 shows a sample installation.

Finally, fills may be compacted to reduce the risk of road failure when crossing small drainages. Compaction increases the density of

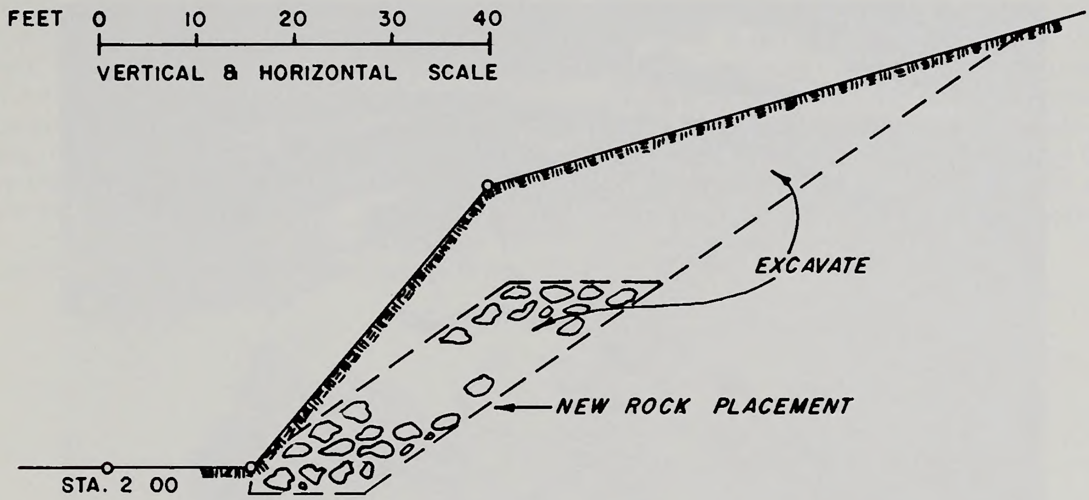


Figure 48. A sample design for a rock buttress to provide support for a failing slope. Note the toe of the buttress is below the subgrade to provide the best support for the buttress.

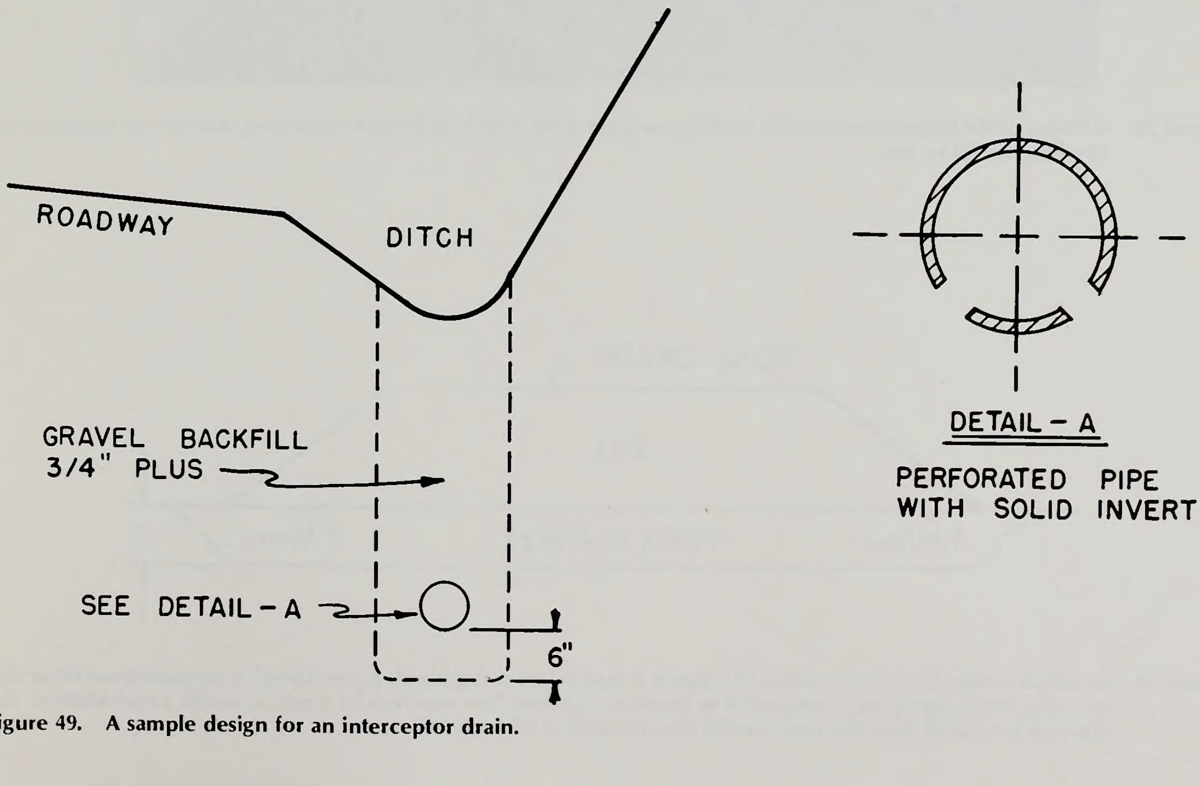


Figure 49. A sample design for an interceptor drain.

the material, reduces the pore space, and thereby reduces the adverse effect of ground water. The foundation material under the proposed fill should be evaluated as part of the design process to determine if this material will support a compacted fill without failure. Figure 50 shows a foundation failure of deeply weathered, very wet material of the Otter Point Formation beneath a compacted fill. Roads may often be built across gentle slopes of incompetent material with a high ground water table by over-excavating the material, placing a thick blanket of coarse material, then building the road on the blanket as shown in Figure 51. The coarse rock blanket distributes the weight of the roadway over a larger area, and it also provides better drainage for ground water under the road.

Figure 50. A failure of the foundation material under a compacted fill. Note how the soft underlying material was squeezed out allowing the fill to sink.

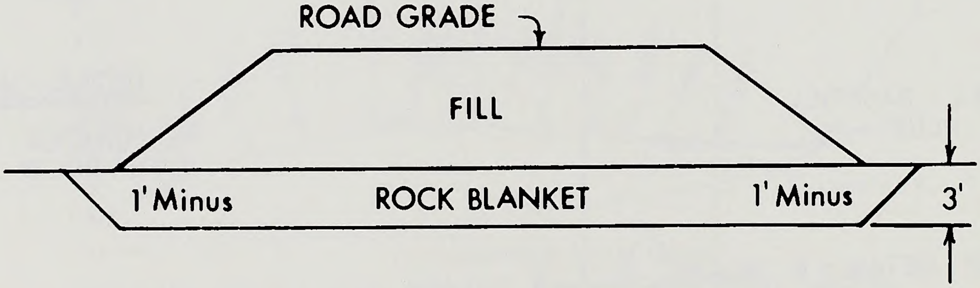


Figure 51. A sample design for a rock blanket to support a road fill on soft ground. A core trench is excavated and filled with one-foot minus riprap and compacted in layers not greater than one foot in depth to create a rock blanket. The material excavated from the core trench may be used as fill material.

4. Soil Creep

The foregoing types of slope failures are relatively fast moving rockfalls, rockslides, debris avalanches, debris flows, and slumps. Many of these slope failures may be preceded and followed by soil creep, a relatively slow moving type of slope failure. Soil creep may be a continuous movement on the order of less than one foot per decade (Bailey). The indicators of soil creep may be subtle, but foresters and engineers must be aware of the significance of this type of slope failure. Soil creep at any moment may be immeasurable but when the effect is cumulated over many years, soil creep can create stresses within the soil mantle that may approach the limit of frictional resistance to sliding and/or the cohesive shear strength along a potential shear failure surface.

Soil creep is particularly treacherous in conjunction with debris avalanches because the balance between stability and failure may be approached gradually over a number of years until only a heavy winter rain or minor disturbance is necessary to trigger a slope failure. Soil creep also builds up stresses in potential slumps such that even moderate rainfall may start a slow earthflow on a portion of the slope. Depending on the particular conditions, minor movement may temporarily relieve these stresses (and thereby create sags or bulges in the slope), or it may slightly steepen the slope and increase the potential for a major slump during the next heavy rain. The point to remember is that soil creep is the process that slowly changes the balance of forces on slopes. Areas that may be stable enough to withstand high seasonal ground water levels this year may not be able to five years from now.

CONSTRUCTION OF STABLE ROADS

Construction of stable roads requires not only a basic understanding of regional geology and soil mechanics but also specific, detailed information on the characteristics of soils, ground water, and geology where the road is to be built. This section presents techniques on road location and the soil and vegetative indicators of slope instability and high ground water levels. Personnel from the Bureau of Land Management and the Forest Service have been interviewed concerning road construction methods on specific geological materials. Their comments and recommendations are incorporated in the following section.

BEDDED SEDIMENTS

The bedded sediments vary from soft siltstone to hard, massive sandstone. These different geologic materials, together with geologic processes and the effect of climate acting over long periods of time determine slope gradient, soil, and the rate of erosion. These factors also determine the particular type of slope stability problem that is likely to be encountered. There are four slope stability problems that are associated with distinctive sites within the bedded sediments. The first two stability problems are found on standstone bedrock and will be designated as Types I and II. The third and fourth problems are found on deeply weathered siltstone and the sandstone-igneous rock contact.

1. Standstone — Type I

Type I slope stability problems are found principally on the Tyee and Yamhill Formations. Type I sites are characterized by sharp ridges

with steep slopes that may show a uniform gradient from near the ridgetop to the valley bottom. The landscape is sharply dissected by numerous stream channels; these channels may become extremely steep as they approach the ridgetop. Headwalls (bowl-shaped areas with slope gradients often 100 percent or greater) may be present in the upper reaches of the drainage. The headwall is usually the junction for several emphemeral streams that can cause sharp rises in the ground water levels in the soil mantle during winter storms. It is quite common to note ground water seepage on exposed bedrock in the headwall even during the summer. Figure 52 shows a block diagram that illustrates the features of Type I sites. This area was taken from the upper Smith River in Oregon as shown on the topographic map in Figure 53.

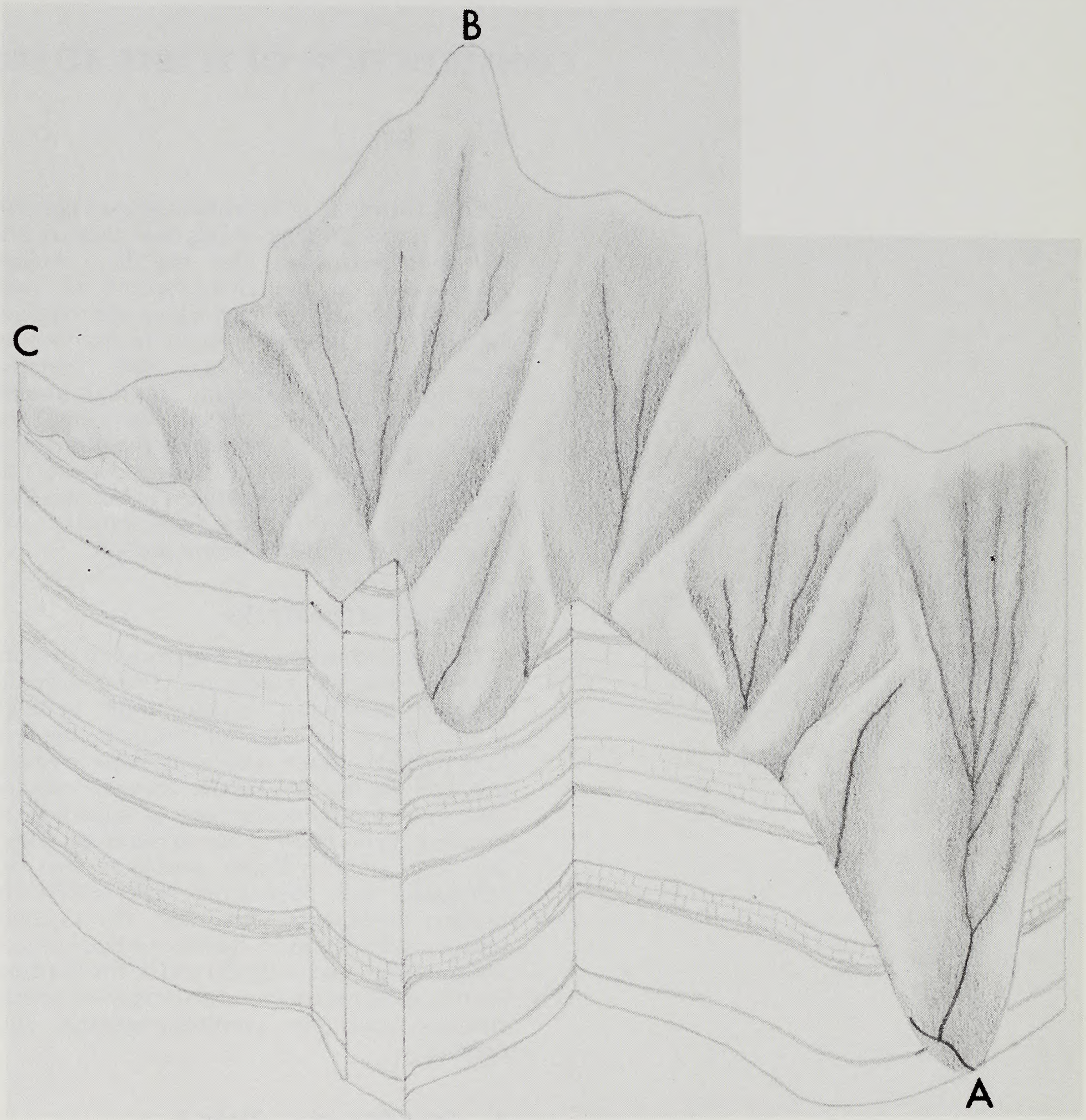


Figure 52. A block diagram of Type I sites, the steep, highly dissected terrain with sandstone bedrock where debris avalanches are the most common kind of slope failure. This diagram was made from the topographic map of the drainage basin outlined in Figure 53. The mouth of the basin is at point A, and two high points are identified as points B and C. The vertical scale of the block diagram is greatly exaggerated. Note the steep headwall areas below point B and above point A. Compare with Figure 64.

Figure 53. A topographic map of the steep terrain with sandstone bedrock where debris avalanches are the most common type of slope failure. The basin outlined in black is shown in the block diagram in Figure 52. Points A, B, and C may be used for orientation. Note the characteristic drainage pattern. Compare with Figure 65.

The soils on the most critical portions of Type I sites are coarse textured and shallow, less than 20 inches to bedrock. These soils are considered to be unstable on slopes greater than 80 percent. Where ground water is present, these soils are considered to be unstable on slopes considerably less than 80 percent.

Debris avalanches and debris flows are the most common slope failure on Type I sites, and the headwall region is the most likely point of origin for these failures. Construction of roads through headwalls will cause unavoidable sidecast. The probability is high for even minimum amounts of sidecast to overload slopes with marginal stability and to cause these slopes to fail (Figures 54 and 55). Observers often comment on the stability of full bench roads that were built through headwalls without realizing that debris avalanches may have occurred during construction before any traffic moved over the road. Figures 56 and 57 show the potential for damage to life and property from the fast moving and very destructive debris avalanche and debris flow.

Figure 54. Two steep headwalls where material sidecast from roads has caused debris avalanches.

Figure 56. The result of a debris avalanche from a road fill in a headwall in sandstone material.

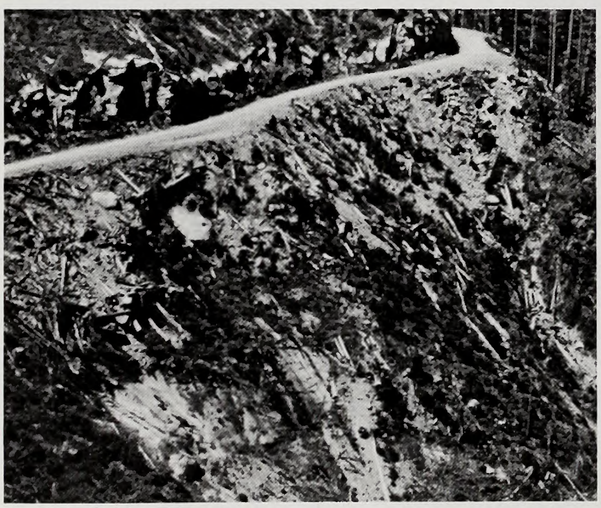


Figure 55. A view of several headwalls with extremely steep slopes and shallow soils.

Figure 57. A debris avalanche came down the stream channel just to the right of the house.

There are certain indicators of unstable slopes for these Type I sites that may be used during road location.

Pistol-butted Trees (Figure 58). These trees were tipped downslope while small as a result of sliding soil or debris, or as a result of active soil creep. As the tree grew, the top regained a vertical posture. Pistol-butted trees are a good indicator of slope instability for areas where rain is the major component of winter precipitation, but it should be kept in mind that deep, heavy snowpacks at high elevations, such as the High Cascades, may also cause this same deformation.

Tipped Trees (Figure 59). These trees have a sharp angle in the stem. This indicates that the tree grew straight for a number of years until a small shift in the soil mantle tipped the tree and the angled stem is the result of the recovery of vertical growth.

Tension Cracks (Figure 60). Soil creep builds up stresses in the soil mantle which are sometimes relieved by tension cracks. These features may sometimes be hidden by vegetation but they definitely indicate active soil movement.

Techniques for proper road location on Type I sites include:

Avoid Headwall Regions. Ridgetop locations are preferred rather than crossing through headwall regions.

Figure 58. Pistol-butted trees may be used as an indicator of active soil movement when the trees were young.

Figure 59. Tipped trees have a sharp angle in their stem.

Figure 60. Tension cracks indicate an actively failing slope.

Rolling the Road Grade. Avoid headwalls or other unstable areas by rolling the road grade. Short, steep pitches of adverse and favorable grade may be included.

Consider the following construction techniques when roads must be constructed across long, steep slopes or above headwall regions where sidecast must be held to a minimum:

Reduce the Road Width. This may require small tractors with narrower blades (D-6, for example) for construction. A U-shaped blade is reported to result in less sidecast than a straight blade, possibly because of better control of loose material.

Controlled Blasting Techniques. These techniques may be used to reduce overbreakage of rock and reduce the amount of fractured material which is thrown out of the road right of way and into stream channels. Appendix E has a more detailed explanation of controlled blasting.

Endhaul Material. Endhauling excavated material away from the steepest slopes may be necessary to avoid overloading the lower slopes.

Select Safe Disposal Sites. Disposal sites for endhauled material should be chosen with care to avoid overloading a natural bench or spur ridge and causing slope failure (Figure 61). The closest safe disposal site may be a long distance from the construction site but the additional hauling costs must be weighed against the damage caused by failure of a closer disposal site with a higher probability of failure.

Figure 61. A disposal site for endhauled material was located to the left of, and below, the road. Note the cracked and sunken grade.

Fill Saddles. Narrow saddles may be used to hold endhauled material by first excavating narrow bench roads below, and on each side of the saddle. The saddle may then be flattened and the loose material that rolls downslope will be caught by the benches. Endhauled material may be compacted on the flattened ridge to build up the grade (Figures 62 and 63).

Culvert Size. Choose a culvert size which will carry the maximum estimated flow volume. The extra size will provide capacity for the unusual storm.

Protect Slopes. Culverts must not discharge drainage water onto the base of the fill slope. Culverts should either be designed to carry water on the natural grade at the bottom of the fill or downspouts or half round culverts should be used to conduct water from the end of shorter culverts down the fill slope to the natural channel.

Figure 62. Endhauled material may be compacted on narrow ridges.

Figure 63. This ridge road has remained stable through several winters.

2. Sandstone — Type II

Type II stability problems are also found principally on the Tyee and Yamhill Formations. Type II sites have slopes with gradients which range from less than 10 percent up to 70 or 80 percent. The longer slopes may be broken by benches, the ridges are rounded, the drainages are fewer, and the slope gradients are more gentle than those on Type I sites. Figure 64 is a block diagram of Type II topography taken from the topographic map shown in Figure 65 (compare with Figures 52 and 53). Headwalls are rare and small patches of exposed bedrock are only occasionally found on the steeper slopes.

The soils on the gentle slopes have developed over many centuries and are deep (often greater than 40 inches) with a clay content as high as 50 to 70 percent. The soils on the steeper slopes may be as deep as 40 inches but the bedrock is fractured and weathered such that there is a gradual transition from the soil into the massive bedrock. It is these factors of deeper soils, higher clay content, gentler slopes, and a gradual transition to bedrock which makes this terrain more stable than the terrain on Type I sites.

The factors which characterize Type II sites also cause this terrain to have more slump and earthflow types of slope failures. The most unstable portions of Type II sites are the steep, concave slopes at the heads of drainages, the edge of benches, or those locations where ground water tends to accumulate. Road failures frequently involve poorly consolidated or poorly drained road fills and embankments greater than 12 to 15 feet on any of the red clayey soils such as Honeygrove, Blachly, or Jory. The convex portions of ridges are also susceptible to failure. Here the clayey Honeygrove, Peavine, and Blachly soils on the gently sloped ridgetops

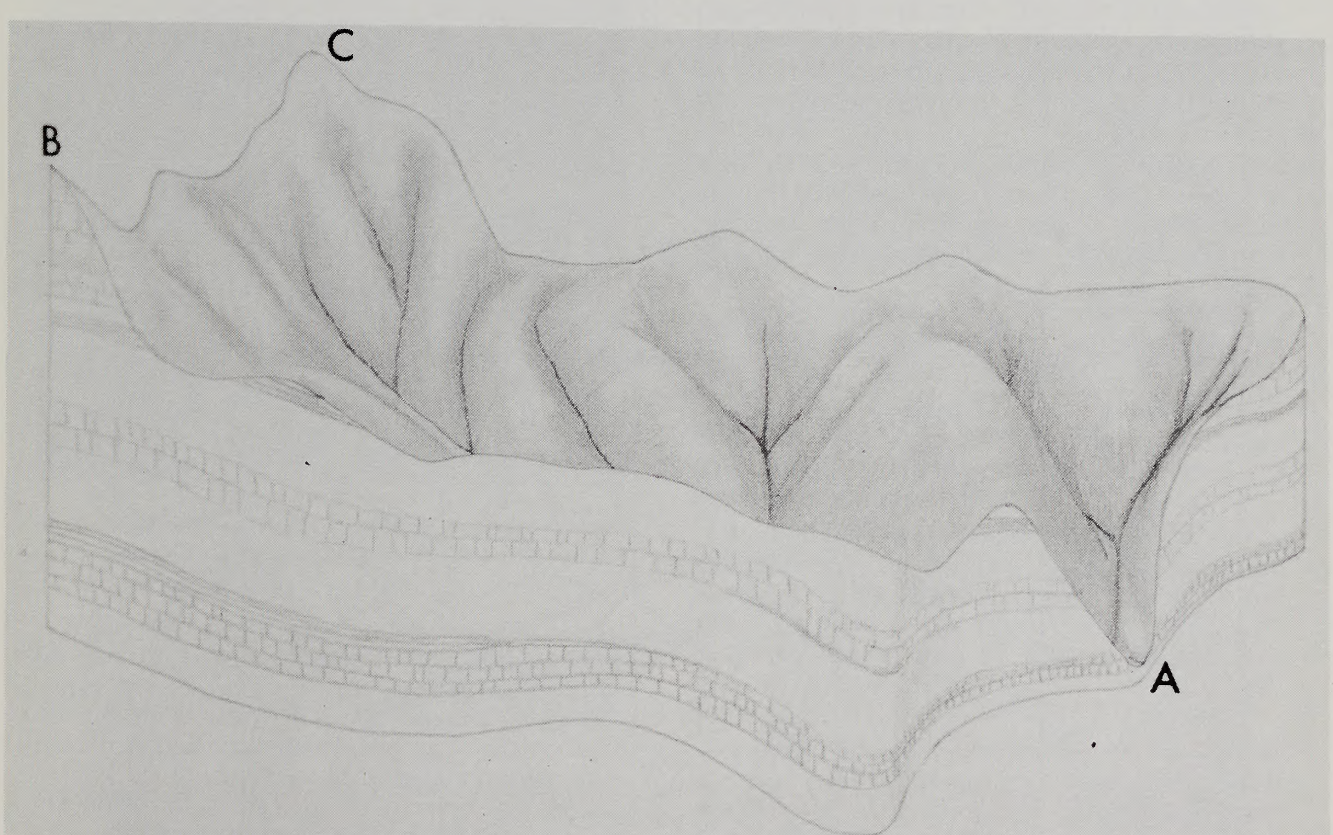


Figure 64. A block diagram of Type II sites. The geologic material is sandstone; the slopes may range from 10% to 80% but the soils are deeper and the ridges are more rounded. Compare with Figure 52. Slumps and earthflows are the most common type of soil failure. This diagram was made from the topographic map of the drainage basin outlined in black on Figure 65. The mouth of the basin is at A, and the high points are at B and C.

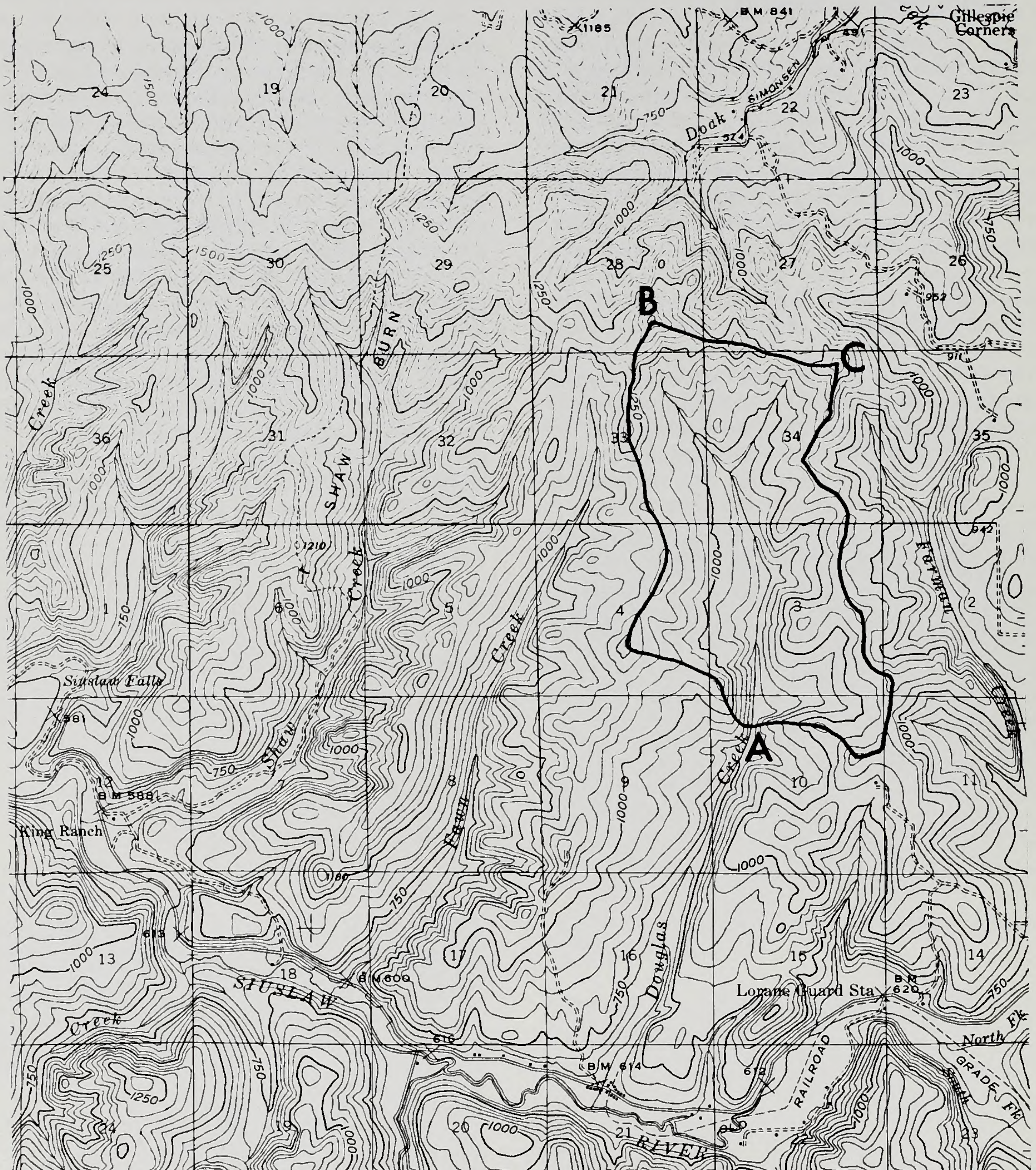


Figure 65. A topographic map of an area with sandstone bedrock, deep soils, and rounded ridge tops. Slumps and earthflows are the most common kind of slope failures. The drainage basin outlined in black is shown in the block diagram of Figure 64. Points A, B and C may be used for orientation. Note the characteristic drainage pattern. Compare with Figure 53.

change to the more shallow, coarser-textured Digger and Bohannon soils on the 60 to 70 percent side slopes. Soil creep also creates tension within the clayey soils at the convex ridgenose where slope gradients may be only 50 percent. Excavation for roads at these points of sharp slope convexity sometimes causes failure of the embankment.

Vegetative indicators of unstable portions of the landscape include mature trees that tip or lean as a result of minor earthflow or soil creep on the steeper slopes. Tipped or leaning trees may also be found on poorly drained soils adjacent to the stream channels. Actively moving slopes may show tension cracks or cat steps (Figure 66), particularly on the steeper slopes.

Figure 66. Cat steps, or small scarps, result from slope failures.

There are several good indicators which may be used to determine the height that ground water may rise in the soil and roughly how long during the year that the soil remains saturated. Iron compounds within the soil profile will oxidize and turn rusty red or bright orange and give the soil a *mottled* appearance when the ground water rises and falls intermittently during the winter. The depth below the soil surface where these mottles first occur indicates the average maximum height that this fluctuating water table rises in the soil. At locations where the water table remains for long periods during the year the iron compounds are chemically reduced and give the soil profile a gray or bluish-gray appearance. The occurrence of these *gleyed* (rhymes with "played") soils indicates a soil that is saturated for much of the year. Occasionally mottles may appear above a gleyed subsoil which indicates a seasonally fluctuating water table above a subsoil that is subject to prolonged saturation. Engineers and foresters should be aware of the significance of mottled and gleyed soils when they are exposed during road construction. These indicators give clues to the need for drainage or extra attention to the suitability of a subsoil for foundation material.

Techniques for locating stable roads on Type II sites include:

Avoid Steep Concave Basins. Avoid locating roads through steep, concave basins where stability is questionable, as indicated by vegetation and topography; ridgetop locations are preferred.

Stable Benches. Benches may offer an opportunity for location of roads and landings, but these benches should be examined carefully to see that they are supported by rock and are not ancient, weathered slumps with marginal stability.

Avoid Cracked Soil. Avoid locating a road around convex ridgenoses or below the edge of benches where tension cracks or cat steps indicate a high probability for embankment failure.

Certain design and construction practices should be considered when building roads in this terrain.

Avoid Overconstruction. If it is necessary to build a road across steep drainages, avoid overconstruction and endhaul excess material to avoid overloading slopes.

Avoid High Cut Embankments. An engineer or soil scientist may be able to suggest a maximum height at the ditch line for the particular soil and situation. Twelve to fifteen feet have been suggested as a rule of thumb estimate for maximum height for a cut bank in deep, clayey soils.

Special Attention to Fills. Fills of clayey material over steep stream crossings may fail if the material is not compacted and ground water saturates the base of the fill. Fill failures in this wet, clayey material on steep slopes tend to move initially as a slump, then may change to a mud flow down the drainage (see Figure 67). To reduce the chance of fill failures, the following alternatives are suggested:

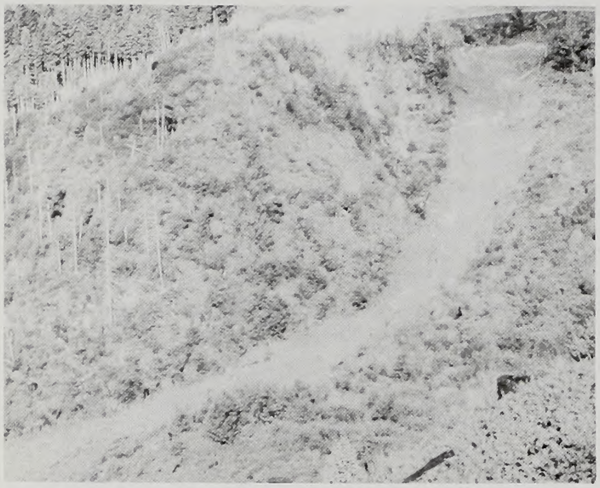


Figure 67. Failures of saturated, clayey fills may begin as a slump, then continue as a mudflow.

Compaction. Compact the fill to accepted engineering standards, paying special attention to proper lift thicknesses, moisture content, and foundation conditions.

Drainage Design. Design drainage features, where necessary, to control ground water in the base of a fill. For example, consider either a perforated pipe encased in a crushed rock filter (Figure 49) or a blanket of crushed rock under the entire fill (Figure 51).

3. Deep Soils Derived from Siltstone

The Nestucca Formation contains considerable siltstone and the stability problem originates in the siltstone that is basically incompetent and easily weathered. Slumps and earthflows, both large and small, are very common when this material is subjected to heavy winter rainfall (as much as 100 inches in the Nestucca River basin). The landscape may exhibit a benchy or hummocky appearance. Slopes with gradients as low as 24 percent may be considered to be unstable in deeply weathered siltstone with abundant water. Many of the higher ridges in the Nestucca Formation are composed of intrusive volcanic rock and the siltstone forms the easily eroded basins bounded by the harder material. Bear Creek drainage, a tributary of the Nestucca River, contains examples of these stability problems (Figure 68).

Vegetative and topographic indicators of slope instability are numerous. Large patches of alder, maple, or other hydrophytes[1] indicate high ground water levels and impeded drainage (see the large patch of alder and big leaf maple in Figure 68). Conifers may be caused to tip or lean by earthflow or soil creep. Slumps cause numerous benches, some of which show sag ponds. Blocks of soils may sag and leave large

Figure 68. Terrain derived from the weathered siltstone of the Nestucca Formation. Note the hummocky slopes, the large slump in the clearcut area (arrow), and the hydrophytes at the right of the photo.

cracks which gradually fill in with debris and living vegetation. The sharp contours of these features soften in time until these cracks appear as "blind drainages" or sections of stream channel which are blocked at both ends. These cracks collect water, keep the ground water reservoir charged, and contribute to active soil movement.

[1]Hydrophytes are plants which are associated with wet soils. The particular species which indicate abundant soil moisture should be locally defined because the species may vary by latitude, elevation, or regional precipitation.

Techniques for locating stable roads through terrain which has been derived from deeply weathered siltstone include:

Indicators of Ground Water. Avoid locating roads through areas where ground water levels are high and where slope stability is likely to be at its worst. Such locations may be indicated by hydrophytes, by tipped or leaning trees, and by mottled or gleyed soils.

Consider Ridges. Ridgetop locations may be best because ground water drainage is better there. Also, the underlying rock may be harder and may provide more stable road building material than the weathered siltstone.

the area is moving, especially at point C where the ground water level is kept at a high level by sag ponds at B.

Special road design and construction techniques for this type of terrain may include the following:

Drainage Ditches. Every effort should be made to improve drainage, both surface and subsurface, since ground water is the major factor contributing to slope instability for this material. Sag ponds and bogs may be drained by ditches excavated by tractor or by ditching powder. Sag ponds and depressions at point A in Figure 69 were connected to the road ditch by

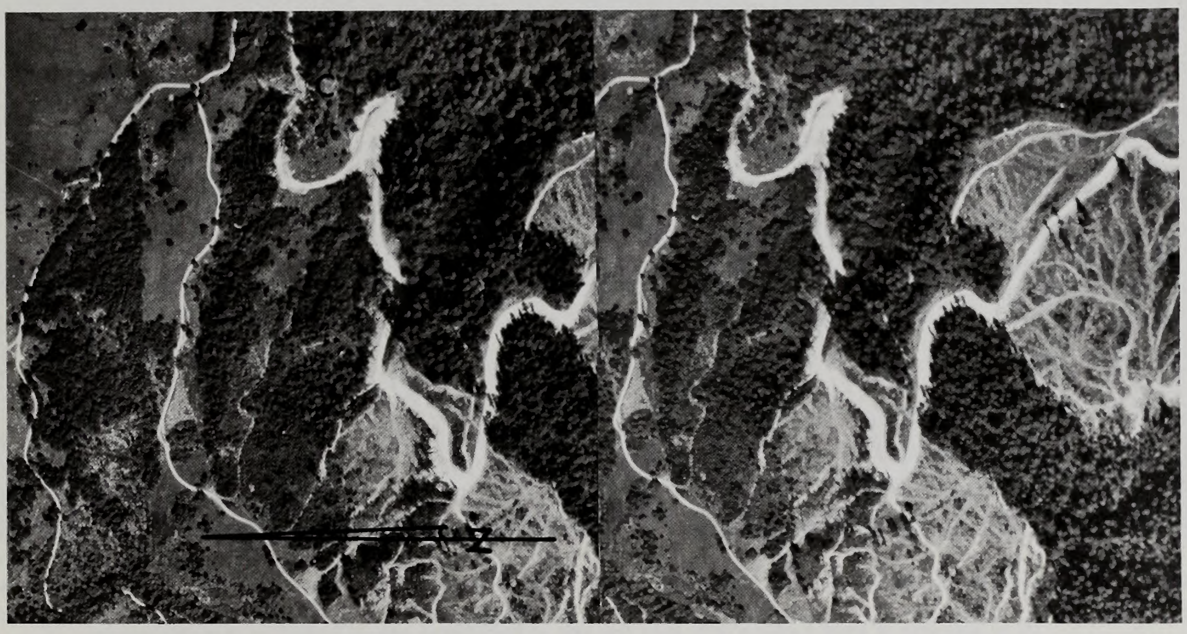


Figure 69. This is a stereogram of a road which was built along a large, old slump in the Yamhill Formation. Point A is at the base of the main scarp. The light-colored area below point B is a disposal site for material from slumps into the road, mostly from point C. Ground water levels are high from point A through B to C. Sag ponds and water-filled depressions are numerous. Ditches connecting sag ponds to the road ditch northwest of point A have improved drainage of surface water and ground water. Ditches were deepened around the disposal site at B and extra culverts were installed to remove water and stabilize these slopes.

Adequate Reconnaissance. Take pains to scout the terrain away from the proposed road location using aerial photos and ground reconnaissance to be assured that the line does not run through, or under, an ancient slump whose stability may be upset by road construction. Figure 69 is a stereogram of a road location across the bottom of a large, old slump (point A in the stereogram) in the Yamhill Formation. Sag ponds and depressions are common within the forest from A to B. Much of

a series of tractor excavated ditches as shown in Figure 70.

Extra Culverts. Extra culverts should be used to prevent water from ponding above the road and saturating the road prism and adjacent slopes. The light-colored area in B in Figure 69 is a disposal site for debris from slumps along the road from point C to the sharp curve. This disposal site hindered the movement of surface water from the ponds and bogs at B. Ditches were deepened and extra culverts were

Figure 70. Ditches can be used to drain sag ponds and depressions.

installed on either side of the disposal site to conduct water more rapidly out of the area and across the road.

Good Road Ditches. Road ditches should be carefully graded to provide plenty of fall to keep water moving and a special effort should be made to keep ditches and culverts clean following construction.

4. Remnants of Sedimentary Material on Steep Ridges of Igneous Rock

The fourth type of special slope stability problem in bedded sediments is caused by remnants of sandstone adjoining ridges of igneous rock, usually diorite. Igneous rock has intruded the bedded sedimentary material and erosion has removed all but scattered patches of this sedimentary material. As a general rule, any contact zone between sedimentary material and igneous material is likely to have slope stability problems. The igneous rock may have caused fracturing and partial metamorphosis of the sedimentary rock at the time of intrusion. Also, water is usually abundant at the contact zone because the igneous material is relatively impermeable compared to the sediments and the sedimentary rock may therefore be deeply weathered.

Figure 71 shows a stereogram of a portion of Prairie Peak and the East Fork of Lobster Creek northwest of Corvallis. The road parallels the sandstone-diorite contact except at the large point at A, the small ridgenose at B, and the large spur ridge at C. The contact zone is exposed at D. Note the alder which outlines the contact zone at B. Figure 72 shows a ground view of this indicator of ground water accumulation. The large spur ridge at C may be relatively stable because of the large mass of material involved, and stability problems may be confined to the immediate contact zone (Figure 73). The arrow above A in Figure 71 indicates the location of a debris avalanche which occurred since these photos were taken.

Special road location techniques for this type of slope stability problem include:

Attention to Contact Zone. Examine the terrain carefully on the ground and on aerial photos to determine if the mass of sandstone is large or small relative to the igneous rock mass. If the sandstone is in the form of a relatively large spur ridge, then the contact zone deserves special attention. The contact zone should be crossed as high as possible where ground water accumulation will be at a minimum. Elsewhere on the ridge of sandstone the stability problems will be the same as for sandstone Type I or Type II.

Consider Alternative Location. If the remnant of sandstone is relatively small, such as a ridgenose, then the entire mass of sandstone may be creeping rapidly enough to be considered unstable and the road should be located above this material in the more stable igneous rock.

Design and construction techniques to be considered are as follows:

Avoid High Embankment. The sedimentary rock in the contact zone is likely to be fractured and may be somewhat metamorphosed as a result of the intrusion of igneous rock. In

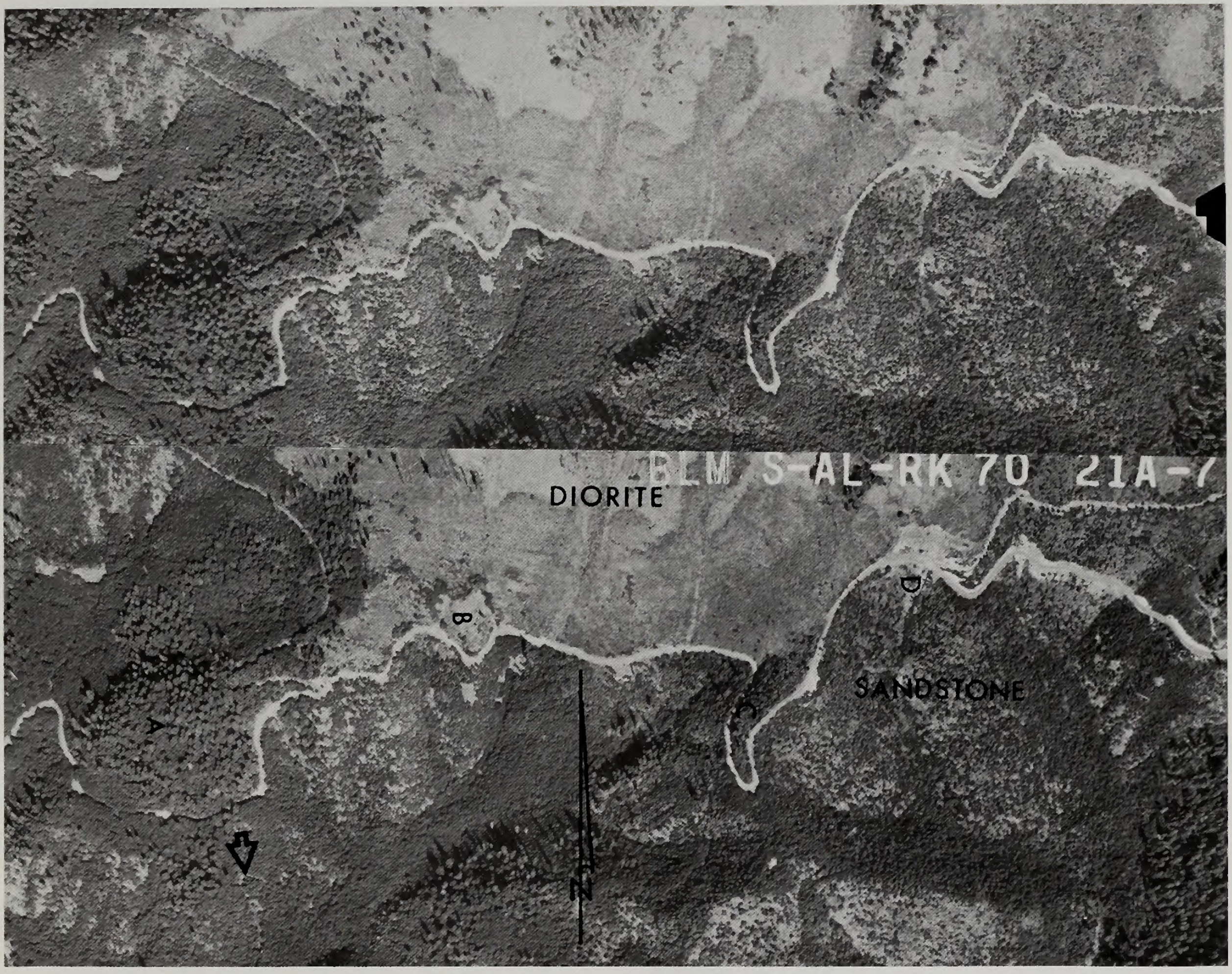


Figure 71. This stereogram gives a contract between diorite and sandstone. The road generally parallels the contact except at A, B, and C. The arrow locates a debris avalanche which occurred after these photos were taken. Ground water accumulation is indicated by the vegetation to the left of B. The contact zone is exposed at D.

Figure 72. Small sandstone ridgenoses on a slope of igneous rock.

Figure 73. Contact zone between sandstone (left) and igneous rock (right).

addition, the accumulation of ground water is likely to have caused extensive weathering of this material. The road cut height at the ditch line should be kept as low as possible through this zone. Support by riprap rock may be necessary if the cut embankment must be high.

Good Drainage. It is good practice to put a culvert at the contact zone with good gradient on the ditches to keep the contact zone well drained. Other drainage measures, such as drain tile or perforated pipe, may be necessary to control ground water.

IGNEOUS ROCKS (EXCEPT GRANITOID ROCKS)

Slope stability problems of the intrusive igneous rocks (gabbro and diorite) usually concern contact with sedimentary rocks, and these were discussed under Bedded Sediments, Subsection 4, "Remnants of Sedimentary Material on Steep Ridges of Igneous Rock." Slope stability problems in granitoid rocks will be discussed in a later section.

The extrusive igneous rocks (basalt and andesite) are often interbedded with pyroclastic rock (Figure 74); therefore, the slope stability problems of the extrusive igneous rocks and the pyroclastics overlap and will be considered together.

Figure 74. Red breccia (bottom) overlain by basalt.

1. Extrusive Igneous Rocks (Basalt and Andesite)

Extrusive igneous rocks generally provide better road building material than the bedded sediments. While the rock itself may be quite stable, the material may weather into deep soils with a potential for slumps and earthflows. The upper slopes are frequently steep with shallow soils while the lower slopes usually have thick soil mantles as a result of erosion from the upper slopes together with deep weathering. Subsurface flow usually moves rapidly downslope through the thin soil mantle that overlies the impermeable bedrock. Much of this ground water appears as springs at the upslope side of benches that are common along the streams in this terrain. The remainder of the ground water seeps through the deep soils to the streams. Road cuts often expose soil layers buried as a result of ancient slope failures and ground water may move laterally along these layers and appear as springs in the cut face after road construction (Figure 75). These local wet spots are a frequent source of small slumps.

Massive slumps are often found on the landscape, and sag ponds and hummocky terrain tend to keep the ground water levels high in these areas. Tipped and jackstrawed trees, together with hydrophytes, are the vegetative indicators of abundant ground water. Other features of concern are the steep slopes between the benches and the streams. Erosion of the base of these slopes results in a gradual steepening of the slope with a corresponding increase in the potential for failure. Tension cracks are occasionally seen at the edge of these benches as the soil pulls apart.

The alternating layers of pyroclastic and extrusive rocks can create serious stability problems. For example, if basalt overlies pyroclastic material, then the slumping of the soft pyroclastic rock can cause the collapse of large portions of the rigid basalt as support is

Figure 75. The arrow shows a buried soil layer; pencils show springs.

removed. This situation is discussed in a later section on pyroclastic rocks. On the other hand, if pyroclastic rock overlies relatively impermeable basalt, then ground water will infiltrate the pyroclastic material and move laterally along the contact zone. Excavation for a roadway through this material may leave the contact zone halfway up the embankment (Figure 76). In this case the pyroclastics can slide onto the road, but the height of the contact zone creates an awkward situation for placing rock buttresses or perforated pipe. Geophysical exploration techniques should be used, if at all possible, during road location so that this situation can be avoided by changing the road location. If the road location cannot be changed, then rock buttresses and/or perforated pipe may be placed as the contact between these two materials is exposed during excavation.

The "progressive" slope failure is another type of stability problem that is frequently found in deep soils on steep slopes in extrusive igneous material. This failure starts as an ordinary cut bank slump into a road. The removal of support causes failure of the next block of soil immediately upslope, and so forth, until there is a series of slumps one above the other that may extend to the top of the ridge (Figure 77). Ground water is usually abundant at these locations and engineers should be on the lookout for signs of an impending slope failure that could develop into a progressive slump.

Techniques for selecting the best road locations in this terrain include:

Use of Photos. Careful examination of aerial photos to identify ancient slumps in advance of field surveys.

Figure 76. Pyroclastic materials overlying basalt

Figure 77. A progressive slump is a series of slope failures that may extend to the ridgetop.

Adequate Reconnaissance. Scout the terrain above and below the proposed road to locate springs, depressions, and bogs for estimates of extra drainage design.

Consider Alternate Location. If ground water appears to be a serious problem where a proposed road will cross a bench, then consider shifting the road onto the steeper slope immediately above the bench being careful not to initiate a slope failure by removing toe support. Geophysical equipment may be used to determine the difference in depth of soil at these alternate locations.

Avoid Cracked Soil. Avoid locating a road near portions of slopes with tension cracks. This is an obvious and reliable indicator of a failing slope.

Design and construction techniques which should be considered include:

Use Special Help. Use every available specialist, together with geophysical equipment, to determine the type and magnitude of potential slope stability problems if it is necessary to build a road through a short section of unstable terrain.

Good Drainage. If the potential stability problem involves ground water, then several remedial measures may be considered in addition to drainage of surface water:

Horizontal Drains. Perforated plastic pipe installed in road cuts and fills may be required to control ground water.

Interceptor Drains. Road cuts with zones of ground water which are too extensive to be handled by perforated plastic pipe augered into the road cut may have the water removed by an interceptor drain installed under the ditch.

Embankment Support. Potential slumps in the road cut may require support in the form of a rock buttress. Rock buttresses are especially effective in preventing progressive slumps.

2. Pyroclastic Rocks

The pyroclastics include: 1) tuffs derived from volcanic ash, and 2) breccias, which are coarser textured and contain angular fragments of relatively hard material. Tuff and many of the breccias weather rapidly to clay and this characteristic makes the location of pyroclastic materials particularly important in the construction of stable roads. Tuffs and breccias may occur in isolated pockets, in relatively extensive deposits, or as layers sandwiched between layers of extrusive igneous rock.

Tuffs and breccias can come in a wide variety of colors which range from dark reddish-purple through light yellow to green (Figures 12 and 13). While all tuffs and breccias have marginal stability, the green variety is notoriously unstable. Soil and rock colors seem to provide a local field guide for prediction of the clay group and the relative landscape stability. Research by Paeth, et al., has shown that soils with montmorillonite clays have 2.5Y and 10YR color hues and have been derived from greenish rock. Relatively stable soils with 7.5YR hue have been derived from yellowish and reddish rock.

The relationship between slope failures and pyroclastic material is summarized by Dyrness in a Forest Service study of mass soil movements in the H. J. Andrews Experimental Forest on the west slope of the Cascades in Oregon.

> "In the western Cascades, it has been observed that mass soil movements occur much more frequently in areas of pyroclastic rocks (tuffs and breccias) than in areas where the bedrock is comprised of basalt or andesite. It has also been noted elsewhere that greenish tuffs and breccias are more unstable than their reddish counterparts . . .
>
> "About 94 percent of the events occurred in areas with a tuff and/or breccia substratum, even though only 37 percent of the total area is made up of these rocks. Moreover, 64 percent of the mass movements were on greenish tuffs and breccias which make up only 8 percent of the total area . . . clearly indicating the unstable nature of these materials. Although about 63 percent of the area is underlain by basalt and andesite material, only 6.4 percent of the mass movement events occurred there."

Unfortunately, there are no known techniques for locating pockets of unstable tuffs and breccias within a general region of pyroclastic material. Many of the pyroclastic rocks appear to be relatively competent and stable when they are uncovered during road construction in dry weather. Some of these materials *slake* or crumble when submerged in water and therefore show a significant loss in shear strength during the rainy season. The characteristic of slaking forms the basis for a field test of those materials which show a seasonal loss in shear strength. Figure 78 shows a clod of pyroclastic material. When this material is excavated while dry it apparently possesses considerable strength; it is impossible to crush a dry clod with the fingers. Figure 79 shows the results of a few minutes immersion in water — the clod has completely disintegrated. The slake

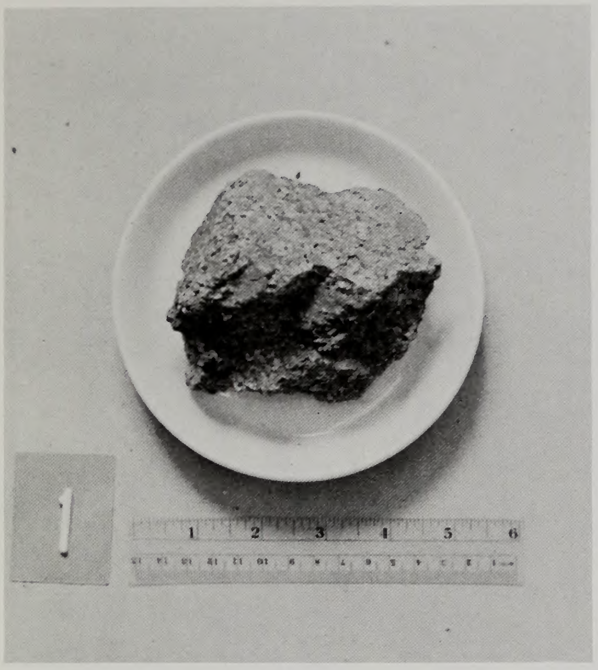


Figure 78. A piece of pyroclastic material. It cannot be crushed with the fingers.

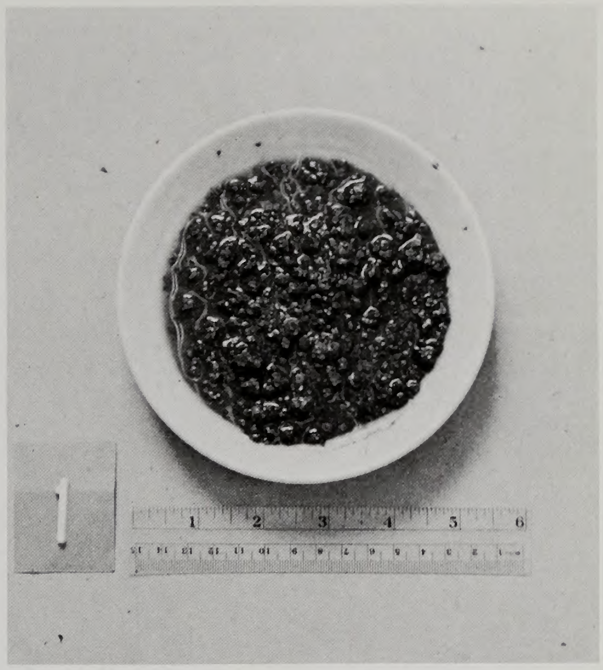


Figure 79. The same clod as in Figure 78. Water has been added, the material has slaked for several minutes and the water has been removed for the photograph.

test may be performed in the field by dropping a dry clod in a cup of water in an upturned hard hat. The use of this field test can provide information for a decision to shift the location of the road or endhaul the excavated material and provide support for the cut slope with riprap.

The various kinds of terrain and slope stability problems that may occur in igneous rocks and pyroclastic material are described in the following illustrations. Figure 80 is a block diagram of an area north of Brice Creek (a tributary of the Row River southeast of Disston, Oregon) which is composed of layers of basalt and tuff. Note how the steeper slopes are associated with the harder basalt and the more gentle slopes with pyroclastics. Figure 81 is a topographic map of this area and the changes in slope caused by alternating materials are readily apparent.

Figure 82 is a stereogram of a portion of upper Thomas Creek, a tributary of the South Santiam River southeast of Mill City, Oregon. This area has a thick layer of pyroclastics capped by extrusive rock. Note the large, circular mass that appears to have slumped away from the higher ridges. There appear to be remnants of the harder cap rock at the summit of the slump mass (point A). There are many other smaller slumps that have created the benchy, hummocky terrain typical of unstable portions of pyroclastic material. There are large slumps, old and new, at B, C, and D, and smaller slope failures near D and E. Note how many of these failures originate in clearcut areas and are not associated with roads. The mature timber in the lower part of the basin contains tipped and leaning trees. Note the large clumps of alder, a locally defined hydrophyte, in the lower drainage at F.

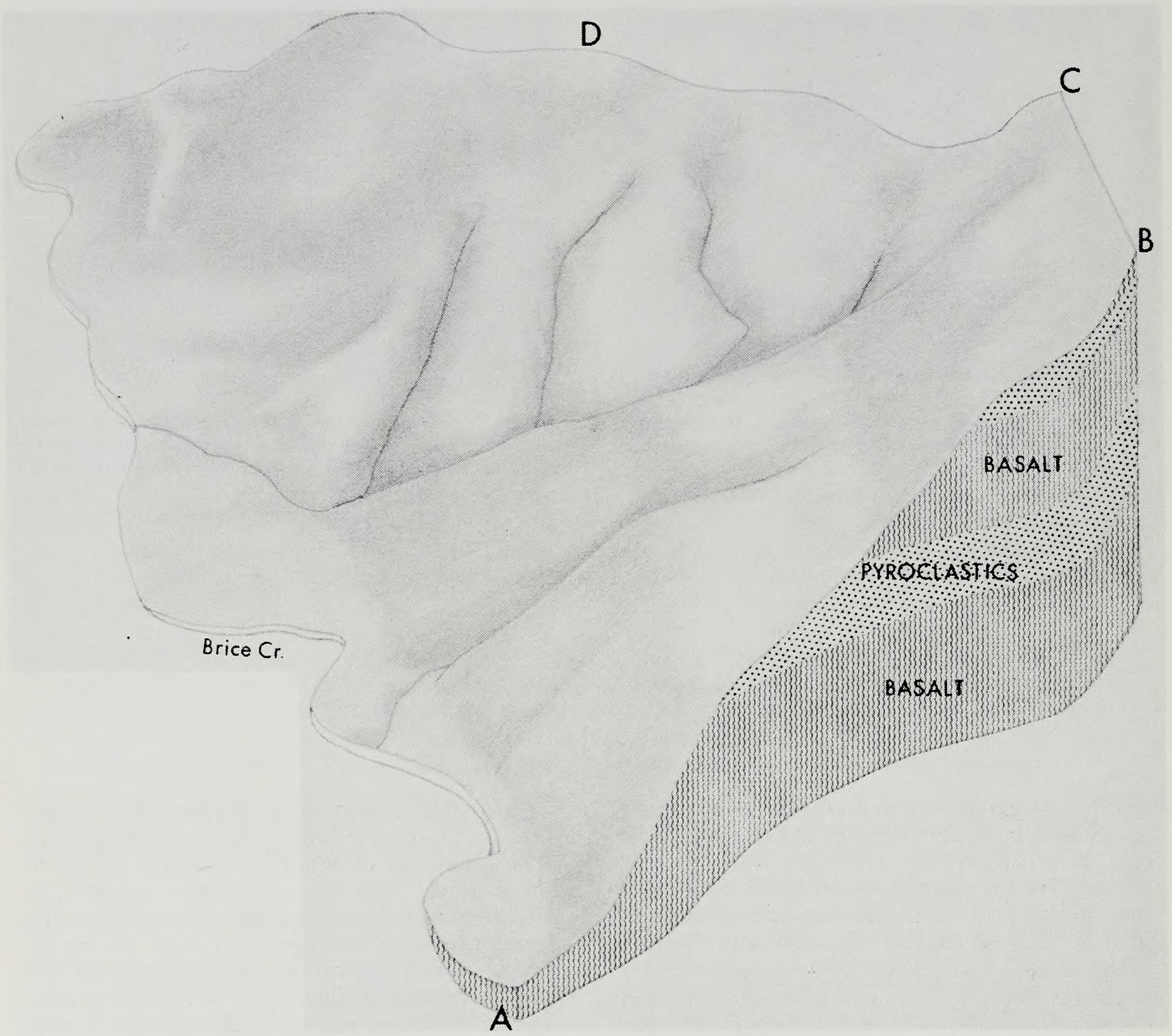


Figure 80. A block diagram of terrain derived from pyoclastic and basaltic materials. The ridges and the steepest slopes are composed of basalt and the gentler slopes are derived from weathered pyroclastics. This diagram was made from the area outlined on the topographic map in Figure 81.

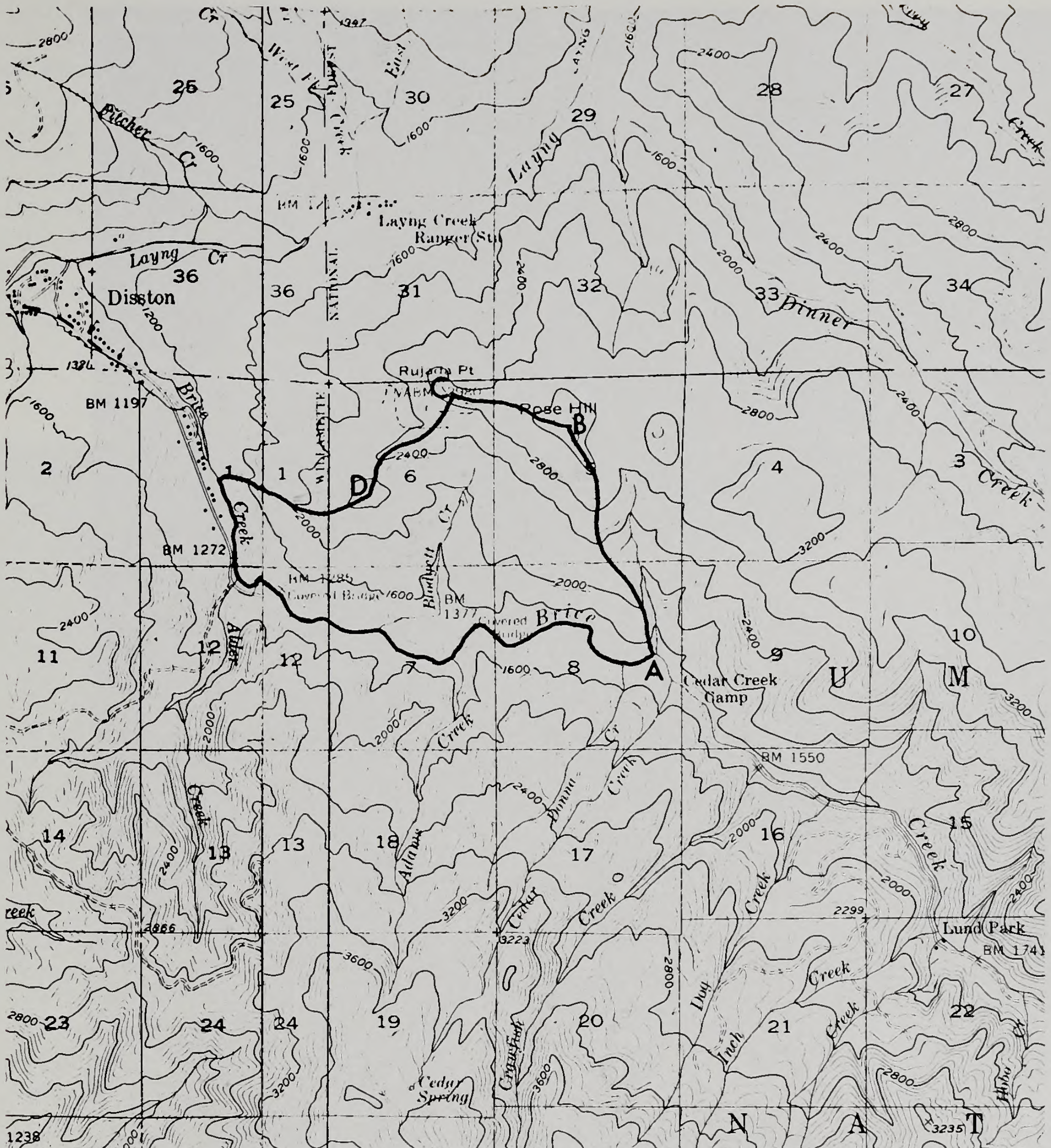


Figure 81. A topographic map of terrain derived from basalt and pyroclastics. The outlined area was used to develop the block diagram on Figure 80; points A to D are used for orientation. Note the many small drainages, the alternating steep and gentle slopes which are typical of this terrain. Slumps are the most common type of slope failure.

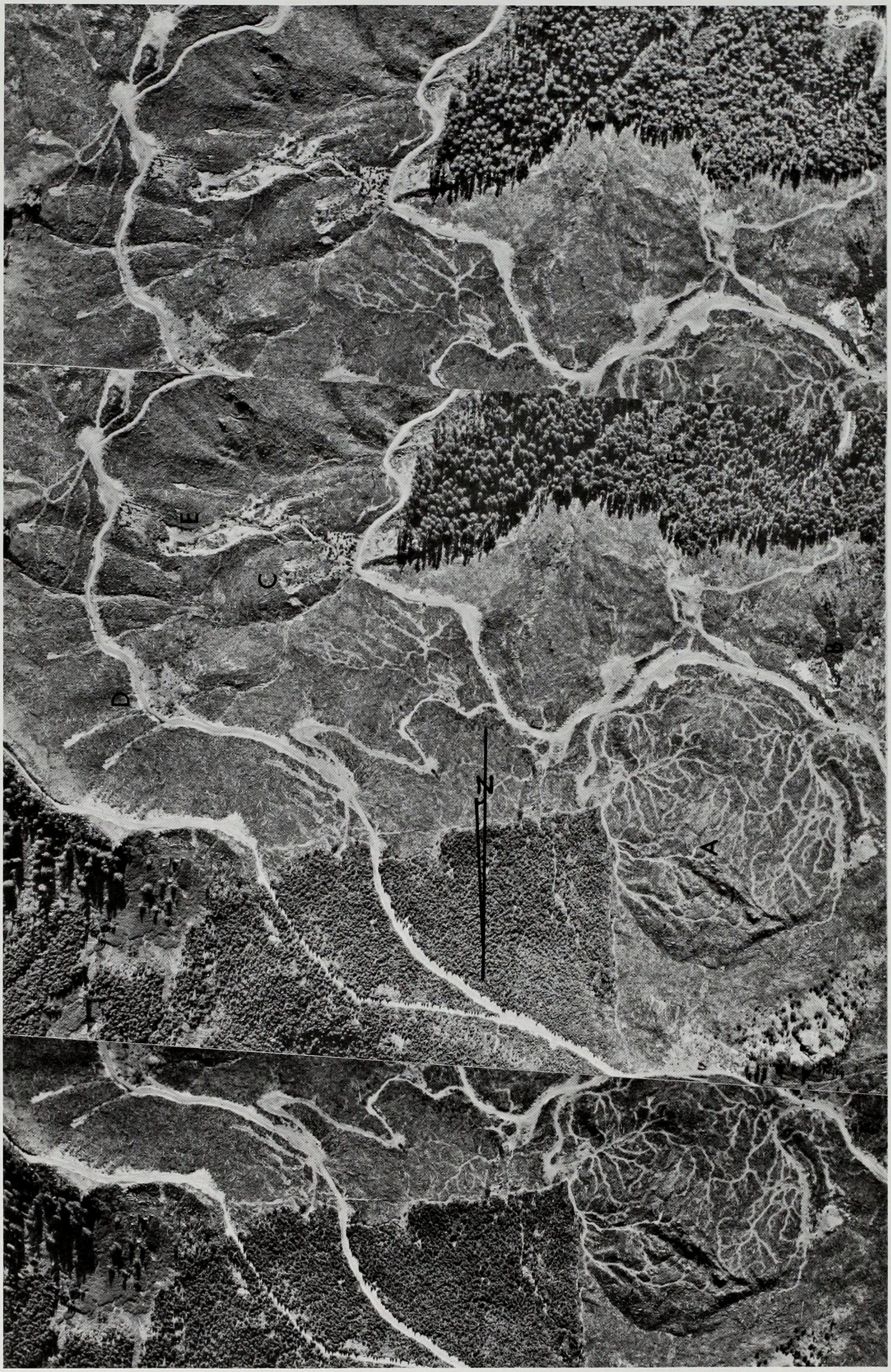


Figure 82. A stereogram of terrain developed from pyroclastics and capped by harder extrusive rock. The large, old slump at A appears to be covered with large pieces of cap rock. Other slumps, old and new, are found at points B to E. Note the hummocky landscape and the presence of hydrophytes at point F.

The Lost Creek drainage is a tributary of the Middle Fork of the Willamette River, south of Lookout Point Dam. This area also contains pyroclastic rocks interbedded between layers of andesite. Figure 83 is a stereogram of this area and it shows outcroppings of the harder rocks from point A around the spur ridge to B. The pyroclastics have slumped away from the ridge at A, and this has left the slump bench at C. The sample of tuff shown in Figure 13 was collected at C. The stream is removing the toe of the slump below C and is therefore gradually steepening the slope and creating an opportunity for more slope movement. The small basin between A and C collects water during the winter rains; this saturates the soil and causes cut bank failures at C. Figure 48 shows a portion of the rock riprap buttress that was designed to support this embankment.

Whitcomb Creek, which flows into Green Peter Reservoir on the Middle Santiam River, is an excellent example of unstable terrain associated with pyroclastic rock. Figure 84 (page 62) is a stereogram that shows an old slump at A.

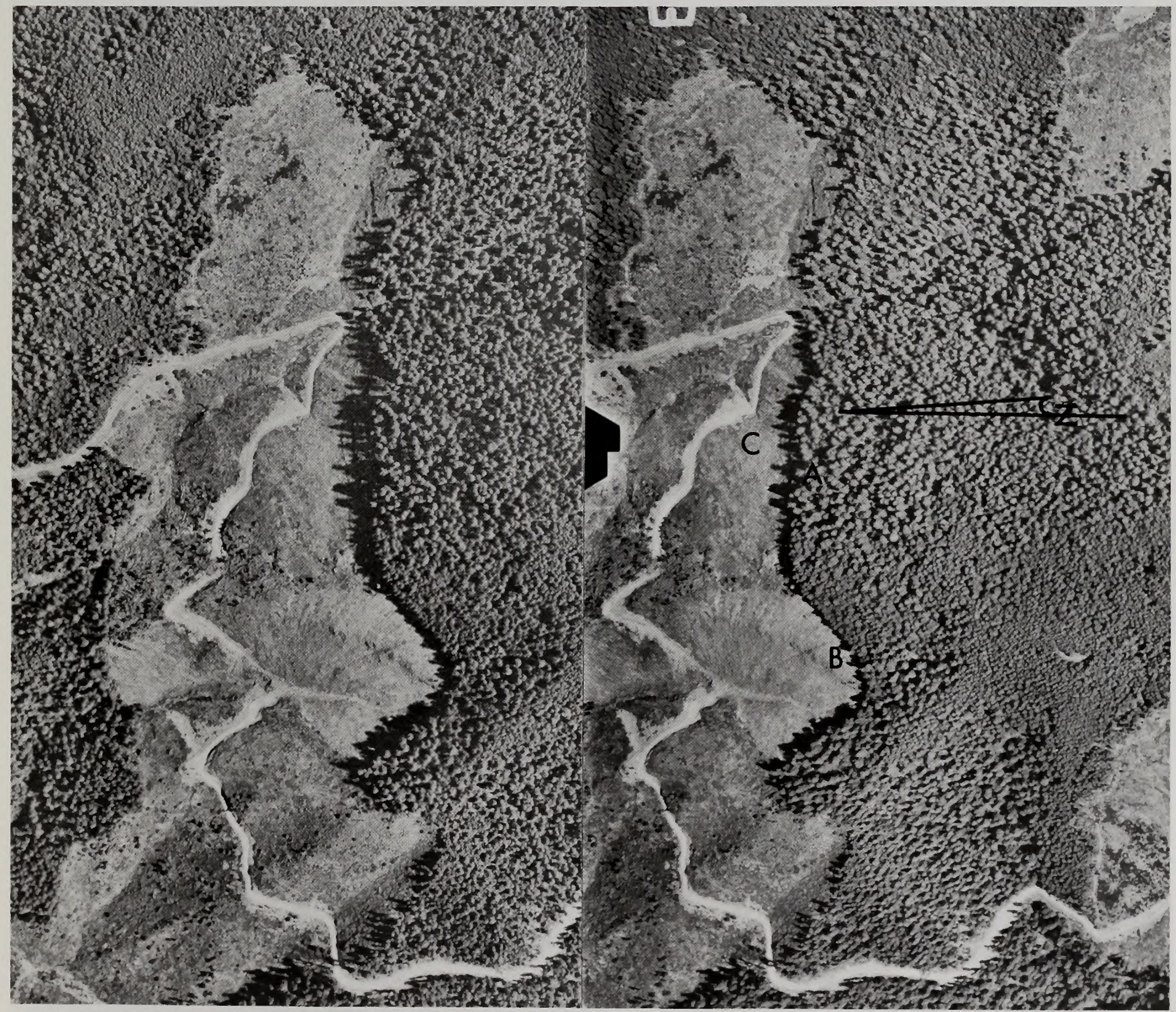


Figure 83. **A layer of thick extrusive rock, which forms cliffs from point A to point B, overlies softer pyroclastic rock. The softer material has slumped to form the gentle slope at point C. Winter rains saturate this slope and lead to cut bank failures at C. The rock riprap buttress shown in Figure 48 was designed to support the cut embankment at point C.**

Figure 84. This stereogram shows unstable terrain derived from pyroclastic rock; there is evidence of active soil movement over the entire photo. An old slump is shown at point A and the dashed line outlines a new slump. Figure 85 shows a ground photo of this slump.

A depression can be seen at the base of the scarp above and to the left of point A. Whitcomb Creek has steepened the slope below this old slump and excavation for the road has removed even more support. This basin failed by slumping after these photos were taken and debris flowed into the creek. The approximate boundary of the new slump is shown on the stereogram. Damage to Whitcomb Creek from the debris flow down the channel was reported to be severe. Figure 85 shows a portion of the new slump and the presence of ground water at the head of the slump is shown by the dark stain on the exposed bedrock.

Location of roads in pyroclastic material should include the following techniques:

Adequate Reconnaissance. Thoroughly scout terrain that has old slumps and hummocks to locate and avoid those areas with acute slope stability problems.

Consider Ridges. A ridgetop location should be considered where the road must traverse areas with a high potential for slope failure or where measures for slope stabilization (drainage, rock buttresses, etc.) would be prohibitively expensive.

Use Benches. Benches on hummocky terrain may provide a relatively stable location provided the road width is reduced so that cut bank height will be an absolute minimum. Figures 86 and 87 show a wide road located on a steep sidehill midway between the stream and a bench. Figure 86 was taken in October and Figure 87 was taken the following January. These figures show that the cut slope is failing and the slump is progressing upward toward the edge of the bench.

Avoid Cracked Soil. In all cases, avoid slopes with tension cracks, cat steps, and small scarps. These indicate that the slope is actively failing and that there is a poor chance to construct a successful road across this terrain.

Consider Alternate Locations. Bogs and swales or other areas with indications of high ground water should be avoided. If it is necessary to cross these areas, scout the surrounding terrain

Figure 85. Ground photo of the new slump outlined in Figure 84. The arrow indicates ground water seeping from the face of the scarp.

Figure 86. Photograph taken in October of new road construction in pyroclastic material. Compare with Figure 87.

Figure 87. Photograph of the same road in January. Note the progressive failure of the slope above the road.

to evaluate the chances of stabilizing the road by drainage or other remedial measures.

Design and construction techniques should be aimed at minimizing the removal of slope support by road excavation and maximizing slope stability through improved drainage.

Avoid Overconstruction. Road design and construction through relatively stable pyroclastics should utilize the minimum possible standard for logging purposes so that slope instability will not be increased.

Avoid Local Unstable Spots. Local areas of marginal stability in material that is otherwise relatively stable may only require a change in grade and/or alignment to keep road width and cut bank height to a minimum.

Be Alert for Unexpected Problems. Equipment operators and contract inspectors should be alert for pockets of unstable material uncovered during road construction. Use the slake test on unfamiliar pyroclastic materials.

Consider Endhauling. Excavated material may need to be endhauled if sidecast will overload the slopes below the road.

Use Special Stabilization Techniques. Short sections of unstable ground may require the following remedial measures, either singly or in combination: single lane road, perforated plastic pipe augered into the slope, rock buttress for bank support, perforated pipe under the road ditch, and drainage of sag-ponds and swales.

Special Attention to Fills. Deep fills in areas of marginal stability should be avoided. Compaction of all deep fills is good practice but the foundation conditions should be evaluated beforehand to see if a rock blanket (Figure 51) may be necessary to support the compacted fill.

GRANITOID ROCKS

Granitoid rock is an extremely variable material. Much granitoid rock weathers to a coarse-textured soil that is low in clay. Those soils that develop in place for long periods may have a clay content of as much as 25 percent or more and may be relatively stable. Those soils that are low in clay have a single-grained structure with little cohesive strength that causes them to be easily eroded (Figures 88 and 89). It is these coarse-grained granitic soils that will be considered here.

The nature of the topography that is typical of deeply weathered, rapidly eroded granitoid bedrock is shown in the block diagram of Figure 90. This topography is from the East Fork of Stouts Creek, a tributary of the South Umpqua River south of Milo, Oregon. Note the steep, highly dissected slopes and the sharp ridges in the block diagram and the characteristic pattern of the contour lines on the topographic map (Figure 91).

Figure 88. This shows the steep, sharply dissected slopes which are typical of much granitic material.

Figure 89. Granitic soils tend to be coarse-grained and very erodible as shown here.

Slopes over 70 percent are considered to be extremely unstable. Slope failures range from very shallow debris avalanches and flows to massive slump failures of road fills. Good drainage of road fills over stream crossings is vital as the coarser material depends almost entirely on frictional resistance to sliding for slope stability. Studies by the Forest Service on the Idaho Batholith north of Boise (as reported by Bailey) shows that 96 percent of the road failures in this granitoid material occurred on slopes of 57 percent or greater. All but two of these failures were on fill slopes and 73 percent occurred in a stream channel or draw. Observations of fill failures in this material show that the failure of one road will often result in successive failures of other roads farther down the same channel as a result of debris flows.

It is a good practice to observe the weathering characteristics of the granitoid rock on older roads in the immediate area before constructing a new road. It has been observed that large pieces of granitoid material from certain areas of the Idaho Batholith begin to crumble a few winters after excavation and exposure to the weather. It is possible that fill failures may be

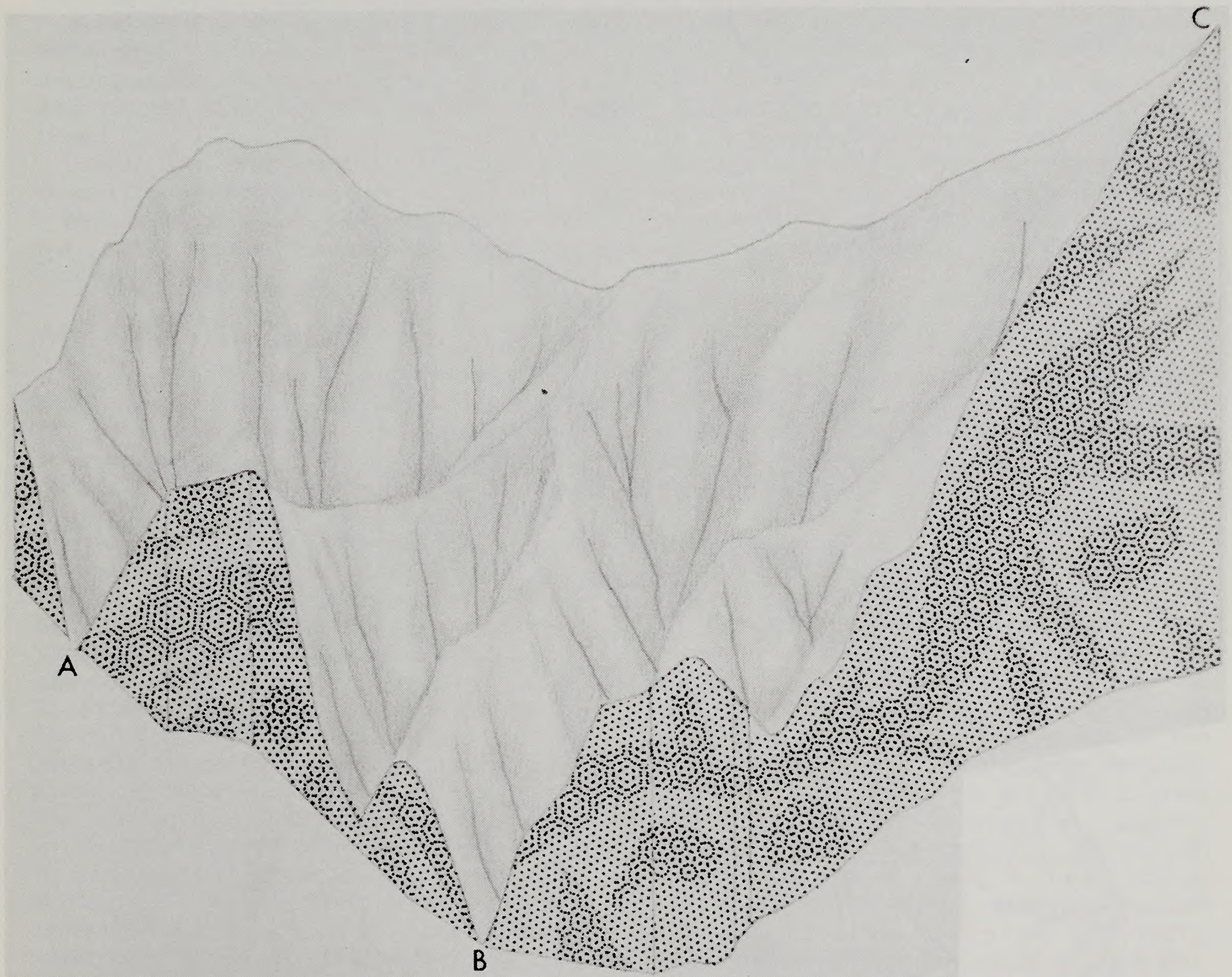


Figure 90. This block diagram of terrain developed from granitoid rock shows the steep slopes, sharp ridges, and sharply dissected slopes that are characteristic of this material. Figure 91 shows the topographic map from which this diagram was developed. Points A, B, and C may be used for orientation.

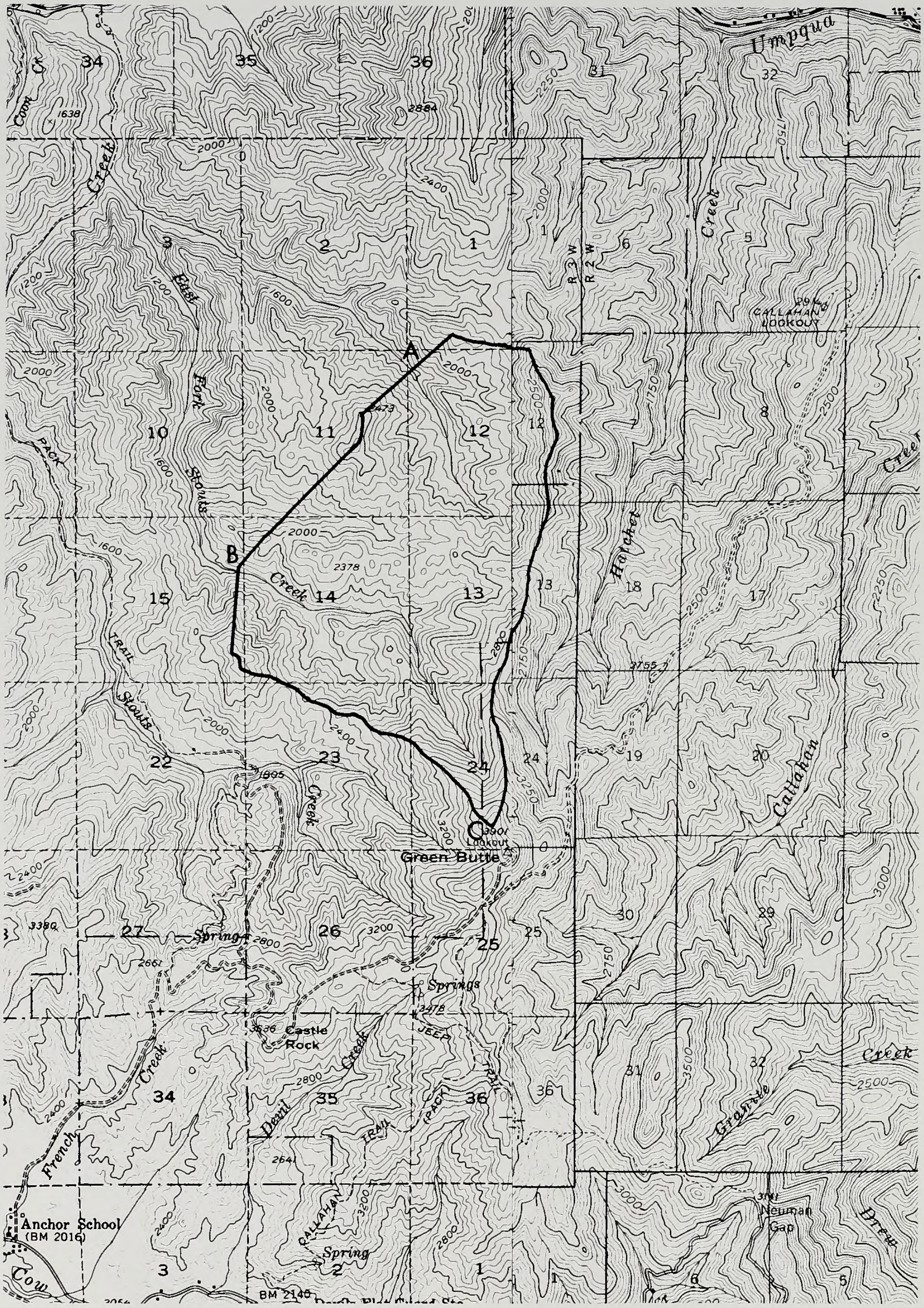


Figure 91. This topographic map was used to develop the block diagram on Figure 90. Points A, B, and C may be used for orientation. The drainage pattern is characteristic of these steep, easily eroded slopes. Debris avalanches are most common on steep slopes, but slumps may occur in deeply weathered material.

attributable in part to the accelerated weathering of this material within the body of the fill.[2]

Much of the following material on location, design, and construction of roads in granitoid rock was developed by the Roseburg District of the Bureau of Land Management and by the Forest Service as reported by Bailey. Road location techniques include:

Avoid Steep Slopes. Locate roads on slopes with a gradient less than 50 percent. Ridgetop road locations are preferred.

Attention to Grade and Alignment. Roll the grade to avoid long, steep side slopes. Do not devote so much attention to alignment that the road does not conform closely enough to the topography.

Avoid Narrow Stream Bottoms. Avoid road locations at the bottom of narrow canyons where road construction will require excavation and removal of support from the base of long slopes.

Design and construction techniques that should be considered include:

Narrow Road Width. Keep the road width narrow. Use the minimum possible standard for logging purposes.

Minimize Earth Work. Sixty-foot radius curves may be necessary to keep earth-moving to a minimum.

Avoid Trash in Fills. Do not allow logs and debris to be incorporated into fills.

Compaction. Compact fills to accepted engineering standards consistent with design standards and material properties.

Culvert Spacing. Design for close culvert spacing to get water out of the ditch before it builds significant erosive power. Frequent culverts will also assure that a large quantity of drainage water will not be discharged onto the lower slope at any one time.

Culvert Alignment. Culverts should be skewed with respect to the ditch line so that water does not have to turn 90° to enter the culvert. Also, consider installing a rock headwall at the culvert entrance to prevent water from bypassing the culvert.

Culvert Gradient. Consider placing culverts on the natural slope to: 1) improve the self-cleaning capability of the culvert, and 2) avoid culvert discharge on the fill slope. Remember to place rocks or other obstacles at the culvert outlet to prevent erosion of the slope.

METAMORPHIC ROCKS

Much of the western portion of the Klamath Mountains and the California Coast Range is occupied by several geologic formations with severe slope stability problems: the Otter Point Formation, together with associated minor formations; the Dothan-Franciscan Formation; and the Galice and Rogue Formations. There are some characteristics relative to slope stability that are common to all these formations. These are the extensive faulting, shearing and weathering that has greatly reduced the competence of the original geologic materials. In addition, slope stability problems for the Dothan, Galice and Rogue Formations are less severe at the north end of their occurrence in Oregon than farther south in California. Part of this variation in stability is caused by differences in the material and partly by regional differences in winter rainfall. Slope stability problems in metamorphic materials may be grouped into several broad categories: 1) blocks of deep, red, clayey soils adjacent to drainages or on steep hillsides; 2) isolated pockets of unconsolidated rock and soil (called colluvium); 3) deeply weathered and easily eroded schists; 4) deeply weathered materials associated with the Otter Point, Dothan-Franciscan, Galice and Rogue Formations.

1. Blocks Of Clayey Soils

Many centuries of soil development have created some very productive red, clayey soils in many areas of southwestern Oregon. These soils were originally developed on gentle to moderate slopes but geologic uplift and subsequent drainage development have left many blocks of this soil on steep hillsides. Downcutting by streams has left some of these soils in a condition of precarious stability. Excavation for roads may remove enough support from these blocks to cause slumps during the winter when the soils become saturated. Figure 92 shows a very large slump along the Galice Road northwest of Grants Pass, Oregon. The magnitude of slope failures such as this makes slope stabilization an impossible task. The best method to prevent these types of slope failures is to avoid locating roads under or through blocks of clayey soils on steep hillsides.

[2]*Personal communication with Rulon B. Gardner, U.S. Forest Service, Bozeman, Montana.*

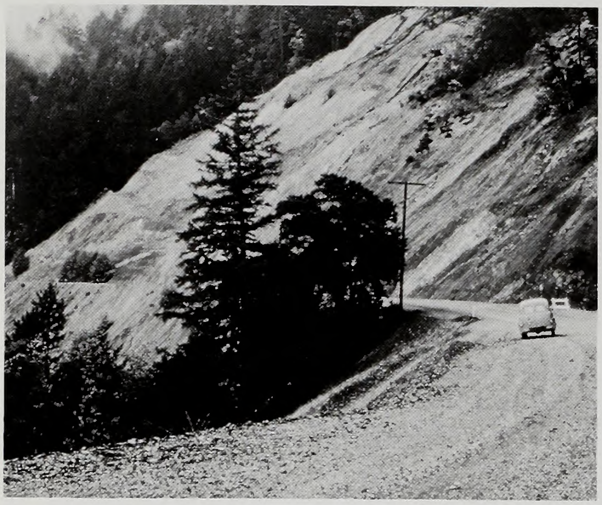


Figure 92. Construction of a road under a mass of clayey soils on a steep slope has caused this material to slump into the road.

The soil depths range from 20 inches to greater than 40 inches and the texture may vary from skeletal (greater than 35 percent coarse fragments) red clays to red clays. The presence of these soils along a proposed road location may be determined from soils maps or by digging down to mineral soil to look for the characteristic red color. Slope instability may be indicated by tipped trees, tension cracks, cat steps, etc.

2. Pockets Of Colluvium

Isolated pockets of colluvium present a different slope stability problem. One theory for the development of these pockets of colluvium is that they represent ancient valleys or slump escarpments which were filled in by soil and rock. Geologic uplift and/or faulting may have lifted masses of material to higher elevations and subsequent development of entirely new drainages has carved valleys which may be at sharp angles to the ancient channels. Road excavation may uncover these deposits of colluvium high on the hillside. This unconsolidated material is usually quite unstable and slumps at these locations are common. Unfortunately, the present-day hillslope offers no clues for locating pockets of colluvium. This material is included in the 370 soil series which is described as a brown, gravelly loam over 40 inches deep. Engineers should be prepared to place rock riprap against the embankment at these locations to provide the support which was removed by road excavation.

3. Easily Eroded Schists

There are two types of schist with severe slope stability problems. The first type includes small pockets of schist along the East Fork of Evans Creek, northwest of Medford within the Applegate Group (see Figure 2). This type of schist occurs near masses of granitoid intrusions, which probably provided the conditions for metamorphism. This black and white schist pictured in Figure 17, weathers rapidly to a gray-colored cohesionless material which resembles coarse-to-fine sand. In fact, it has been reported that an apparently solid rock from a road cut weathered to a pile of "sand" in just one winter. Figure 93 shows the rockfalls which are typical of deep cuts in this material; frost action is particularly effective in loosening the face of road cuts. Control of ground water is essential with the Evans Creek schist because of the cohesionless nature of the soils. Very wet areas may exhibit the slumpy, hummocky terrain of some of the siltstone material; note the leaning tree in the center background of Figure 93.

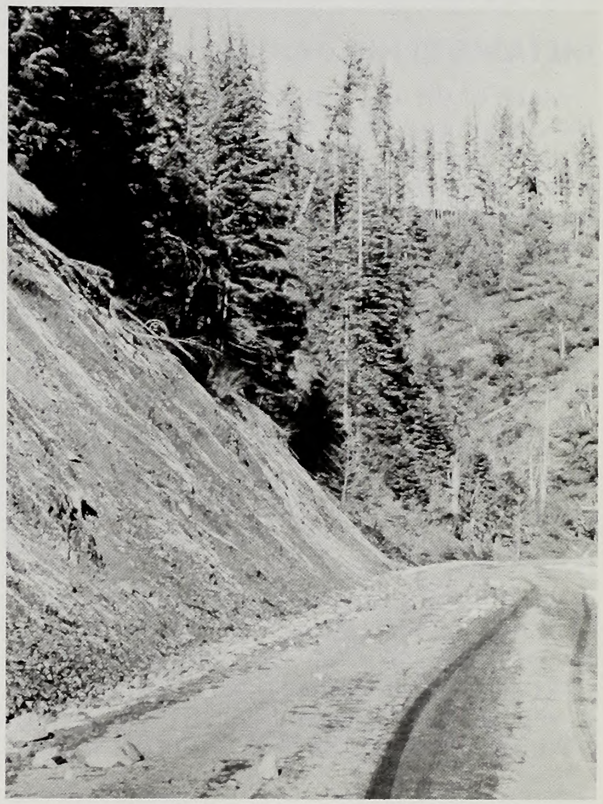


Figure 93. Rockfalls in deeply weathered schist. Note the leaning tree in the background.

Figure 94 shows a sagging road fill and Figure 95 indicates a common cause of road fill failures, both in the schists and in granitoid rocks — impeded ditch drainage as a result of rapidly eroding cut banks. Poor ditch drainage allows water to infiltrate the road prism instead of being carried away. Discharge from culverts can easily erode these soils if the water is allowed to fall on an unprotected fill, as shown in Figure 96. Techniques for road location and construction in the steeper portions of Evans Creek schist are similar to those for the granitoid rocks. For the slumpy, hummocky terrain in the Evans Creek schist use the road location and construction techniques which were suggested for siltstone material under Bedded Sediments, Subsection 3, "Deep Soils Derived from Siltstone," with particular attention to the need for extra culverts.

The second type of schist which requires extra care in road construction is the Colebrook schist (Figure 16) found north and south of the Rogue River near the Oregon coast (Figure 2). The characteristics of this material are variable and range from fractured, relatively stable rock to areas of deeply weathered material which may be very wet with a tendency to slump. As with the Evans Creek schist, the slumpy terrain of the Colebrook schist should be treated similar to siltstone material. Colebrook schist is very erodible and poor culvert design, spacing, and maintenance may result in deeply eroded ditches or massive failures of the road fill.

Figure 94. A cracked and sagging road fill in schist material.

Figure 95. Impeded ditch drainage caused by small slumps.

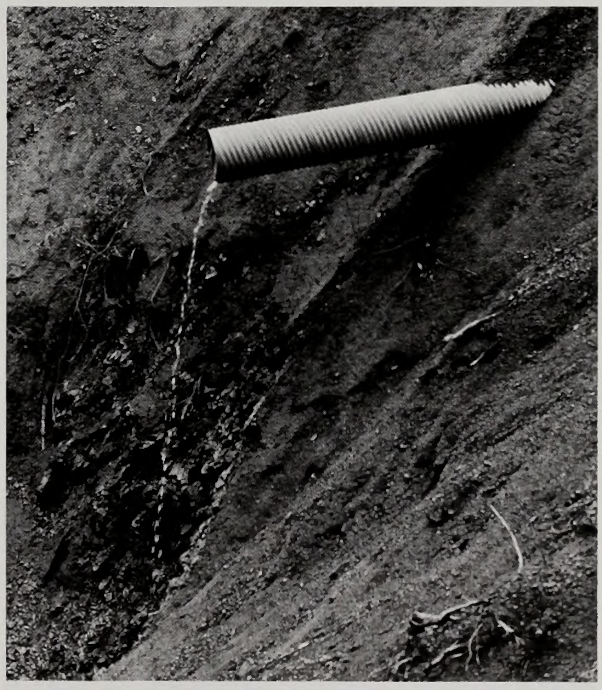


Figure 96. Weathered schists are easily eroded. Culverts should not discharge water onto unprotected fills and slopes.

4. The Otter Point, Dothan-Franciscan, Galice and Rogue Formations

A. The Otter Point Formation contains roughly 30 to 40 percent black mudstone interbedded with fine sandstone; 20 to 30 percent each of coarse sandstone and conglomerate; volcanic rocks make up the remainder (Dott). Pillow lavas, flows, breccias, and tuffs comprise 10 to 15 percent of the Otter Point in the Sixes River area (Baldwin and Hess, 1971). There are also small outcroppings of serpentine scattered throughout the formation.

A typical sample of the slope stability problems frequently encountered in this formation is found in the Baker Creek area, a tributary of the South Fork of the Coquille River near Powers, Oregon. A relatively dark, hard sandstone forms a boundary for the basins which are filled with deep, weathered sandstone and mudstone. The landscape is typical of mélangé terrain with hummocky slopes caused by old slumps and earthflows (Figures 97 and 98). Road failures are usually slumps which remove large portions of the road fill, or settlement of entire sections of the road caused by failure of the underlying material to support a compacted fill (Figure 50), or by slumping of the slope above the road.

Many of the techniques used for locating roads through materials from the Otter Point Formation are covered in the sections on "Deep Soils Derived from Siltstone." In addition, there is another indicator of unstable terrain that is peculiar to the Otter Point:

Avoid Clay Subsoils. There is a soil series within the area occupied by the Otter Point Formation that has a bluish-colored clay subsoil 20 to 30 inches below the surface. The bluish-gray color indicates a gleyed soil and a high water table for much of the year. This soil provides poor foundation material for road construction and is often referred to locally as a "blue clay." Soils scientists can provide information on what portions of the terrain this soil series may be found.

Construction techniques to be considered for materials derived from the Otter Point Formation are similar to those for "Deep Soils Derived From Siltstone" with some important differences:

Support Road Cuts. Many slumps from road cuts start where small drainages intersect the road. Ground water is abundant at these points and riprap may be needed to support the soil material and provide adequate drainage.

Use Rock Blankets under the Roadway. Fills across small drainages and across short sections of marshy ground may require overexcavation

Figure 97. Typical mélangé terrain of the Otter Point formation.

Figure 98. Note the hummocky terrain caused by slumps and earthflows.

and installation of a coarse rock blanket to support the roadway.

Investigate Piling to Support the Roadway. Timber piling was used extensively in the western United States several decades ago to support logging railways. Figures 99 and 100 show a railroad trestle constructed of Port-Orford-cedar in the early 1920s. Portions of this trestle crossed wet unstable terrain as shown here. These pilings are still upright and do not appear to have shifted significantly. It may be possible to design sections of roadway supported by pilings to traverse those marshes or other types of difficult terrain where drainage or rock blankets are not feasible.

Figure 99. Remains of logging railroad trestle constructed in the late 1920s.

Figure 100. Pilings are of untreated Port-Orford-cedar.

B. The Dothan-Franciscan Formation contains sandstone, mudstone, siltstone, and some volcanic rocks in varying proportions throughout its range. In the extreme northern end, the Dothan Formation consists principally of a relatively stable gray sandstone. Portions of the Cow Creek area south of Roseburg have been serpentinized by intrusions of serpentine and ultramafic rocks (Figure 2) and have some severe slope stability problems. Farther south, the amount of thinly bedded black mudstone and siltstone within the Dothan increases to a 4 mile wide zone east of the mouth of the Rogue River (Dott). Beginning about 4 miles east of Brookings, the Dothan Formation is composed of about 60 percent black mudstone and siltstone (Dott). The Franciscan Formation in northern California consists of about 75 percent massive, dark greenish-gray sandstone which weathers to gray, reddish-brown or buff (Rockey and Bradshaw).

The Franciscan Formation is extensively faulted and sheared in northern California. Shear zones of crushed rock vary from a few feet to several miles in width. They tend to be very wet, and contain black or bluish-black material with gravelly clay or a silty mixture of soil or rock (Rockey and Bradshaw).

There is a large fault in northern California, the South Fork Mountain Fault, which separates the Franciscan Formation from the Galice and Rogue Formations to the east. Apparently shear zones associated with faults are relatively rare east of this major fault and shear zones in the Galice and Rogue tend to be found along the boundaries of intrusions of ultramafic rock and serpentine (Rice as quoted in Rockey and Bradshaw).

Techniques for locating roads through the most stable terrain include:

Observe the Dip of Shale Beds. The Dothan-Franciscan Formation has some thick shale layers that are folded and tipped. These beds are stable when they dip into the slope but where the beds lie parallel to the slope, they collect water and weather to a silty clay texture, and slip very easily. Extensive areas of this material often develop a hummocky landscape.

Avoid Shear Zones. Shear zones are identical to shale beds in color and texture. The most important difference is that shear zones cut across bedding planes within the formation and may appear vertically along canyon walls (Rockey and Bradshaw).

Avoid Unstable Grassy Glades. Many glades or open grassy areas are found within the Dothan-Franciscan Formation. Stable glades have a smooth unrumpled appearance on aerial photos with no dark patches caused by lush grass during the dry season (Figure 101). Stable glades also generally have yellowish, brownish or reddish subsoils (Rockey and Bradshaw). Unstable glades may be identified on aerial photos by small slips, minor earthflows, and patches of sedge or grass which are still green during the dry season. These sites usually have grayish to blackish-gray

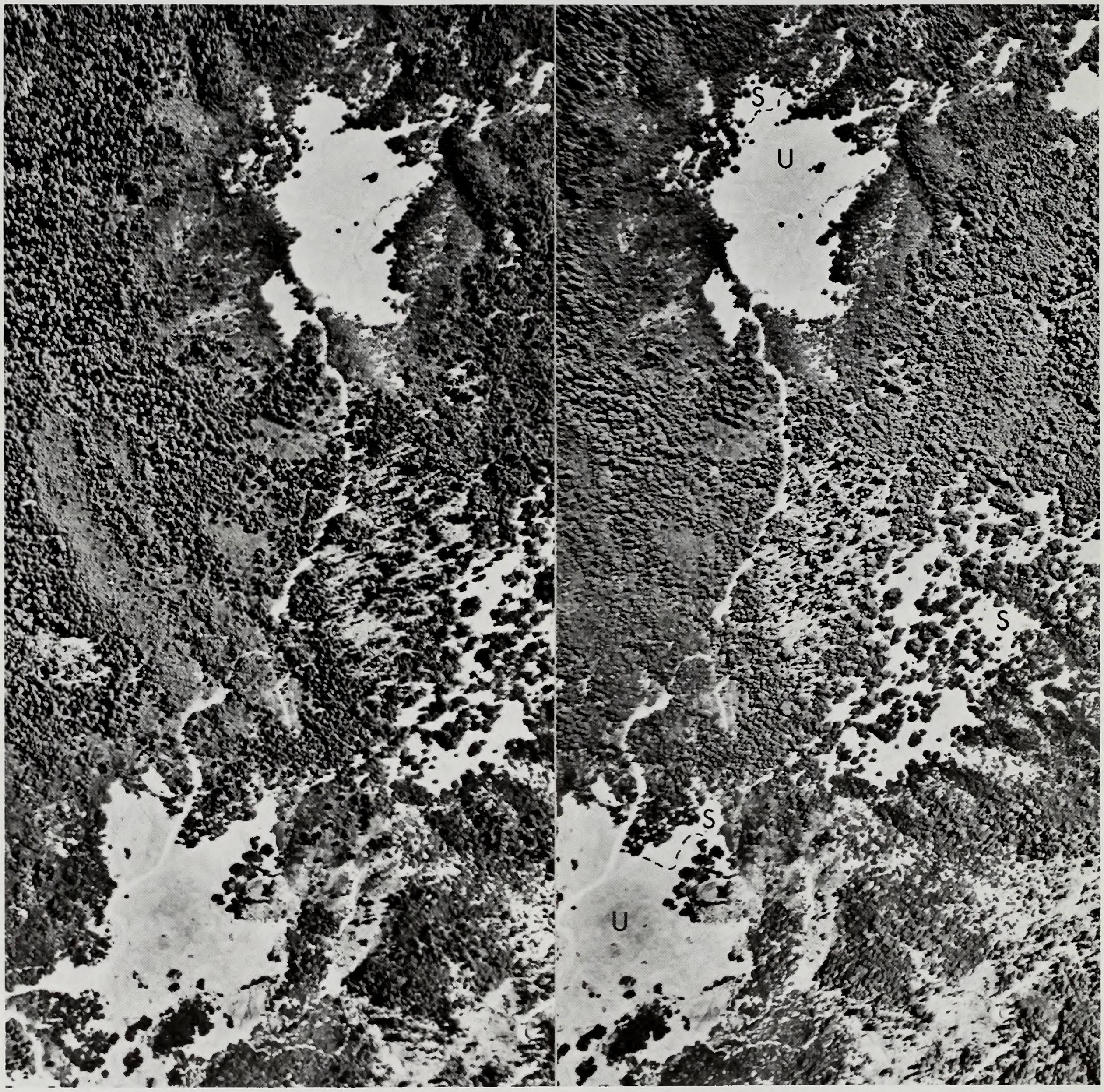


Figure 101. Adapted from Figure 4 of Rockey and Bradshaw. This stereogram shows typical glades that are found on the Dothat-Granciscan Formation. These photos were taken in late August during the dry season. Areas designated "U" are unstable glades with hummocky terrain and darker toned patches caused by green vegetation growing on moist soil. Areas identified by "S" are stable glades with relatively smooth terrain and a light tone caused by cured vegetation.

subsoils (Rockey and Bradshaw). Even low cuts through unstable glades may be difficult to stabilize.

C. The Galice and Rogue Formations will be considered together in this section. There are two types of rocks within these formations: metasedimentary and metavolcanic. The majority of the metasedimentary rocks consist of slate and phyllite with interbedded sandstone. The slates are dark-gray or black and the phyllite is generally light colored with a silky sheen on cleavage surfaces. This material is more highly metamorphosed farther south and some of the phyllite appears black when moist, and shiny gray when dry; this is especially subject to slumps and slides (Rockey and Bradshaw). The metavolcanic rocks include meta-andesite, meta-basalt, tuffs, flow breccias, and thin layers of black slate. Meta-andesitic rocks are the most abundant, appear grayish-green to dark green, and most are tough, massive rocks (Rockey and Bradshaw).

The sharp folding of metamorphosed material, both sedimentary and volcanic, has created V-shaped canyons, steepsided slopes and narrow ridge tops. Portions of the major rivers in northern California, such as the Klamath and Trinity rivers, occupy narrow canyons with little or no flood plain. Here the hillslopes plunge steeply into the rivers. The large floods of 1964-65 undercut many of these steep slopes and started slumps and slides which are gradually extending themselves up the slopes (Figure 102). Road locations which are planned for slopes which face major streams should be checked to see whether natural slides may intercept the road sometime in the future. If so, some adjustment in the planned road location may be necessary.

Techniques for road location within the Galice and Rogue Formations should include:

Become Familiar with Local Indicators of Unstable Areas. Points A, B, and C on Figure 103 illustrate how topographic and vegetative

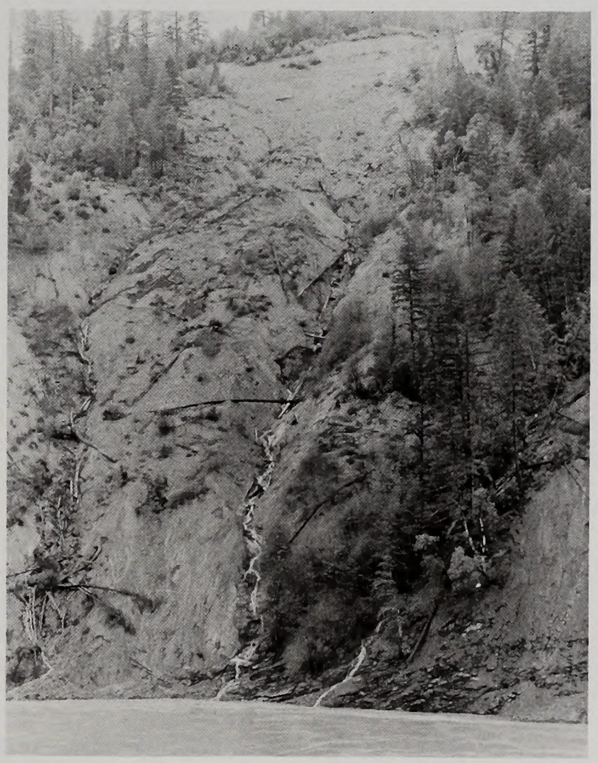


Figure 102. A large natural slide along the Klamath River in northern California. This slide started when the river undercut the slope during the floods of 1964-1965. This slide has progressed until it is threatening the stability of the road shown at the top of the photo. Note the exposed slate just above the water.

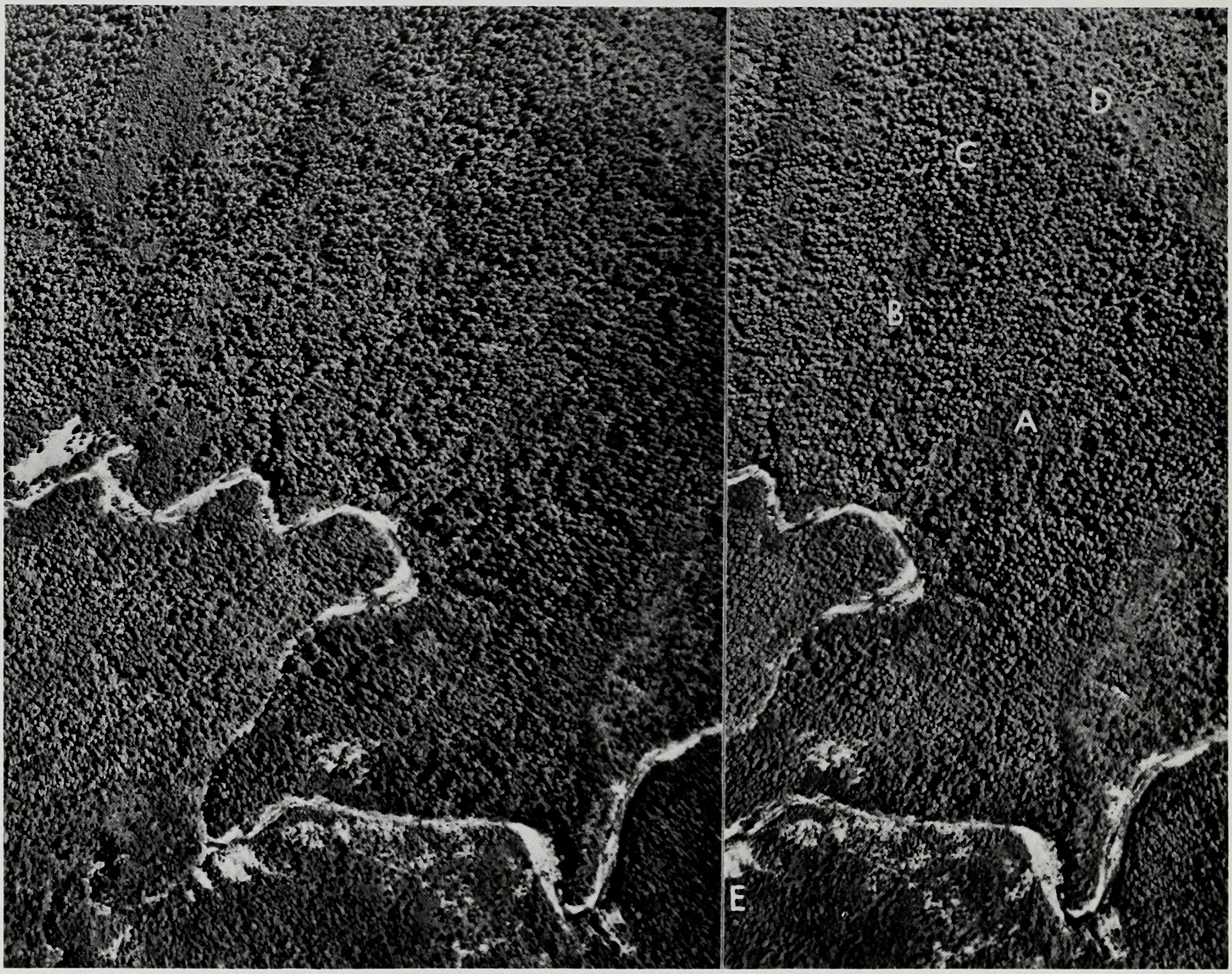


Figure 103. Adapted from Figure 3 of Rockey and Bradshaw. This shows some of the stability problems associated with the slate portion of the Galice Formation. Point A indicates a bowl-shaped slide area with a high proportion of young growth Douglas fir. B shows an unstable draw in an open Douglas fir stand. C indicates a typical open Douglas fir stand on unstable slate material within the Galice Formation. The open canopy is apparently caused by frequent windthrow on the shallow, wet soils. A saddle in the ridge is seen at D. Note the evidence of instability below the saddle. The stream at E has evidently undercut the slope and caused the debris slide at that point.

indicators on aerial photos may be used to locate suspected unstable areas within slate material of the Galice Formation in northern California. In this locality the density and composition of Douglas fir stands offers clues to the stability of the slope as indicated in this quotation from Rockey and Bradshaw:

> "(A Douglas fir-shrub stand) may appear to be a pure Douglas fir stand on the photos. It is characterized by a dense fir overstory showing many openings, as though about one tree in ten had been removed. (Actually, the openings may be caused by windthrow of trees infirmly rooted in the unstable soil.) Understory vegetation may be shrubs and/or ferns. Dense patches of young Douglas fir may mark recent land movements."

Slide areas may show Douglas fir stands with a high proportion of young trees (point A, Figure 103). Care should be used to distinguish these unstable areas from other more stable areas whose stands of young Douglas fir may be caused by fire.

Check Aerial Photos for Active Natural Slides. It is important to check proposed road locations

on aerial photos that are taken several years apart to determine if there are natural slides that are extending themselves upslope. These slides frequently originate where streams have undercut slopes (point E, Figure 103).

Utilize Stable Stream Terraces. Some streams have cut down through deposits of soil and gravel and have left stable terraces or benches along one or both sides of the stream. Terraces may be either level or slope toward the stream. Small isolated terraces or benches resembling old rotational slumps may be found at higher levels on the hillslide. Old rotational slump masses slope toward the hillside and sag ponds may be seen at the base of the scarp.

Some general techniques for constructing stable roads through the Dothan-Franciscan, Galice, and Rogue Formations include:

Keep Road Width to a Minimum. Roads that are unncessarily wide result in high cuts which, in turn, increase the potential for slides and slumps. Wide roads also produce extra amounts of sidecast material which may overload the slope below the road. These considerations are extremely important on the long, steep slopes that are typical of many metamorphic materials. Many of these slopes are at their maximum stable slope gradient and there are few, if any, breaks in slope from ridge to stream.

Provide Necessary Cut and Fill Support. Where roads are required to cross long, steep slopes, it may be necessary to provide support for cuts and fills to prevent progressive slides and slumps. Rock buttresses may be used for low cuts on moderate slopes. Steel or concrete cribbing may be necessary to support high cuts and to support the road section where it crosses small, steep drainages.

Provide Support and Drainage for Shear Zones. Shear zones are especially unstable and every effort should be made to design and install structures to drain ground water from these zones. Support may range from rock buttresses to cribbing, and drainage may range from riprap to interception ditches to perforated pipe drains in the cut bank.

ULTRAMAFIC ROCKS AND SERPENTINE

Ultramafic rocks and serpentine are materials whose particular slope stability problems vary from the relatively dry (30 inches to 50 inches) annual precipitation) interior portions of southern Oregon to the wet (80 inches to over 100 inches annual precipitation) coastal climate of extreme southwestern Oregon and northern California. In the drier areas the slope stability problems appear to be those associated with fault zones, as discussed in earlier sections. Soil scientists report that crushed and fractured rock may extend from several hundred feet to one-quarter mile or more on each side of the ultramafic or serpentine intrusion. When ground water is present in these zones the terrain tends to exhibit a slumpy, hummocky appearance as a result of failure of the material and the weathering of the serpentine to clay.

The presence of extensive areas of ultramafic materials and serpentine may be indicated by vegetation composition and density that is characteristic of these areas. These materials contain an excess of magnesium and only tolerant plants can grow on these soils when magnesium is present in quantities sufficient to be toxic to many plant species. The major coniferous species under these conditions are Jeffrey pine, with some western white pine and knobcone pine (Franklin and Dyrness). Tolerant shrubs and forbs are pine-mat manzanita (*Arctostaphylos nevadensis*) and beargrass (*Xerophyllum tenax*). Figure 104 shows a typical

Figure 104. Sparse stands of Jeffrey pine often indicate the presence of serpentine bedrock.

hillside of serpentine soils in the drier zone that appears as an open woodland with a sparse growth of grasses and shrubs.

Figure 105 is an aerial photo of a portion of Cow Creek, southwest of Riddle, Oregon. The approximate boundary between the serpentine material and the Umpqua Formation is shown as a dashed line. This delineation was made on the basis of the difference in density of vegetation. This same photo interpretation technique can be used to delineate ultramafic rocks and serpentine in northern California (Rockey and Bradshaw).

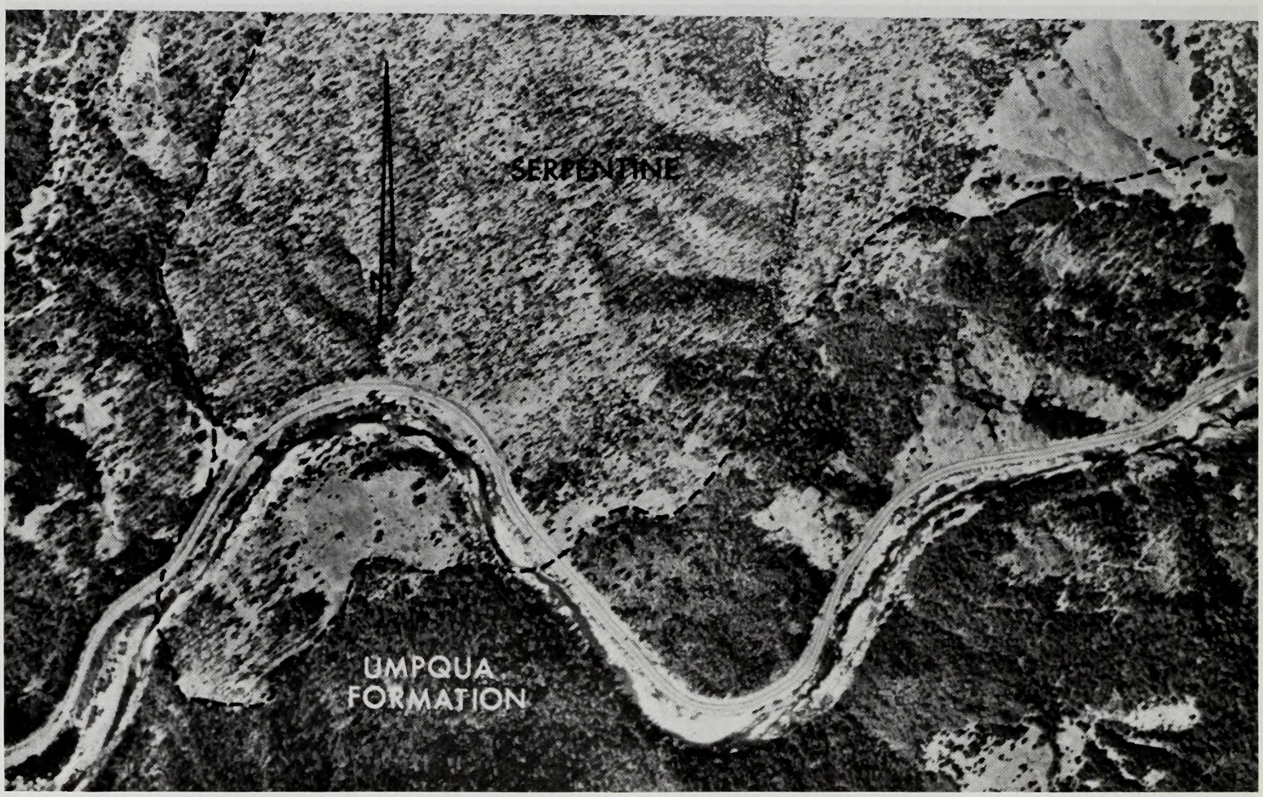


Figure 105. The approximate boundary between serpentine material and the Umpqua formation is shown by the dashed line. This determination is made principally on the basis of vegetation density. Timber on portions of the Umpqua formation have been harvested which accounts for a reduction in vegetation density, particularly in the northwest corner of the photo.

Some of the badly sheared ultramafic rocks and serpentine are quite unstable in the wet coastal climate. There is enough rainfall to cause rapid weathering of exposed serpentine to a sticky clay (Figure 106). Small outcroppings of serpentine occur in other geologic materials such as the Dothan-Franciscan and the Otter Point Formations where vegetative indicators of serpentine may only be a grassy glade in a hillside covered with conifers (Figures 107 and 108). Stable and unstable glades may be differentiated on aerial photos using the technique described in the section on the Dothan-Franciscan Formation. In addition, a ground reconnaissance will show that very small unstable glades will show small slips, tension cracks and cat steps (Figures 107 and 108). Rock riprap for support of the cut bank and control of drainage water may be the only slope stabilization techniques that are possible when roads traverse small areas with serpentine soils.

Some techniques for road location in ultramafic rocks and serpentine include:

Use Maps and Photos. Examine geologic maps and aerial photos to locate intrusions of ultramafic rocks and serpentine.

Indicators of Serpentine. Become familiar with the local indicators of serpentine soils. This should include field identification of key plants, gross delineation on aerial photos, the shape of the terrain, and soils characteristics.

Figure 106. Serpentine often weathers to a sticky clay

Figure 108. Another glade in serpentine material. Roads cut through these steeper hillsides often develop slumps which gradually become larger unless support and drainage are provided.

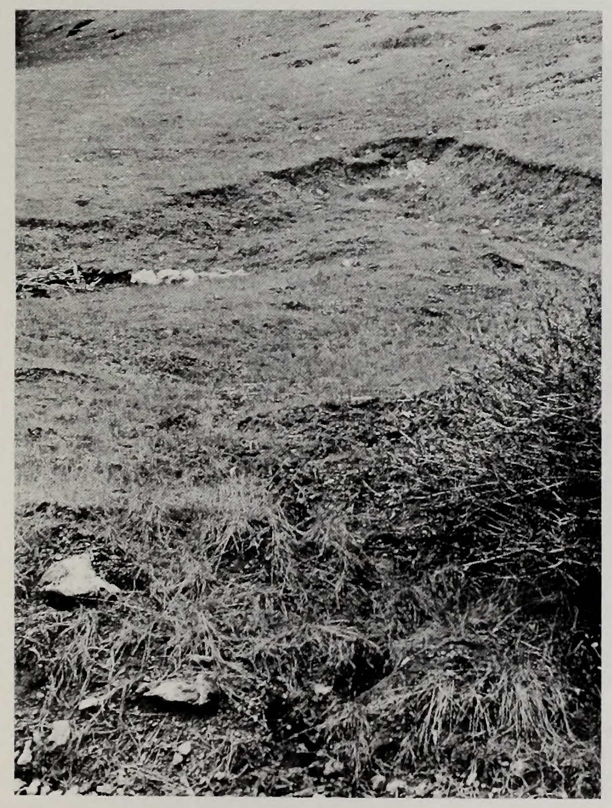


Figure 107. A glade in serpentine material. Note the small seep at the edge of the road at the bottom of the photo and the small slumps on the hillside.

Alternate Locations. Avoid locating roads through basins with extensive areas of unstable ultramafic rocks and serpentine material. It may be impossible, both technically and economically, to stabilize roads once they have been constructed through these basins.

Special Information. Areas which appear to have thin outcrops of ultramafic rocks or serpentine should be examined by specialists to estimate the degree of faulting and/or fracturing, the amount of weathered clayey material which is present, and the amount of ground water which is likely to be present in the winter.

Some construction techniques which should be considered in serpentine material are:

Embankment Support. Be prepared to apply buttresses of heavy riprap rock for slumping cut banks (Figure 109).

Ground Water Drainage. Be prepared to collect and dispose of ground water through extra drainage work as the magnitude of the ground water problems become evident as construction begins. This extra work may include perforated pipe in the cut bank (Figure 110).

Figure 109. Heavy riprap may be used to support slumping embankments in serpentine material.

Figure 110. Perforated pipe may be needed to remove ground water from embankments in serpentine. Note the volume of water issuing from the pipe at the right of center.

MAINTENANCE

Proper road maintenance is extremely important in retaining or improving road stability after construction. Maintenance personnel should have the same understanding of basic geology and soil mechanics as the engineers and foresters who locate, design, and construct the roads. Maintenance personnel who are knowledgeable in these fields can avoid creating unstable conditions on slopes, and they can recognize symptoms of impending slope and road failures so that stabilization techniques can be initiated. Some of the elements of good road maintenance are discussed below.

DISPOSAL OF DEBRIS FROM MASS WASTING

Disposal of debris from avalanches or slumps that falls into the road is a serious problem. Disposal sites which will safely support rock and soil may be a long distance from the slope failure. Nevertheless, such safe disposal sites should be sought out and used. Never take the expedient approach and dump soil, rocks, and debris into drainages or any other location where this material will increase the potential for slope failure. Figure 111 shows the result of a debris avalanche in a steep headwall in Tyee sandstone. The rubble, which extends from the lower part of the photo to the road which circles the top of the headwall, is a result of disposal of debris from cut bank failures elsewhere along the road. In this case, the headwall is primed for another destructive debris avalanche — all it needs is a series of heavy winter rains to raise the ground water level.

Figure 111. Rubble from road clearance has been deposited in this steep headwall. This increases the potential for another debris slide.

SYMPTOMS OF ACTIVE SLOPE MOVEMENT

Maintenance personnel (and also foresters and engineers) should be alert to the signs of impending slope failure or poor drainage, such as sagging of the cut bank, subgrade, or fill, trees which begin to tip or lean, the appearance of new or accelerated ground water discharge from the cut bank, and standing water in ditches. Slope and road failures may be prevented if prompt action is taken to correct the cause of these symptoms of active slope movement.

GOOD CULVERT PRACTICE

Culvert outfall should not be discharged onto fill slopes in a manner which can erode the slope or decrease slope stability through saturation of the fill.

SUMMARY

It is hoped that this publication has emphasized that construction of stable roads in western Oregon and northern California is a complex task that requires a background of knowledge from many fields. The construction and maintenance of stable roads will require many decisions, from the early planning stages through routine road maintenance, and each of these decisions must be based upon an understanding of the basic principles of geology and soil mechanics.

A realistic philosophy towards the construction of stable roads is given by Bailey:

> "It should be realized that not all landslides can be prevented. Probably, the exceptionally large landslides are primarily controlled by the rate of geologic erosion and are largely uninfluenced by activities of man, but smaller earth movements initiated by man's activities are, in turn, at least partially preventable or controllable by man. Included in this concept is the decline of the view of a landslide as an "act of God" and the growing realization that many landslides are initiated by "acts of man." Increasingly, the land manager is faced not only with the increased possibility of landslides, but also with greater economic losses and greater chances for injuries and deaths when landslides occur. In addition, he must necessarily be aware of the effect of management activities in unstable areas on the sustained yield of high quality water because of the increasing need for improved water management. Furthermore, he must consider the effect of management on landslide activity which tends to reduce the soil mantle. The importance of soil loss is apparent when its very slow recovery rate is considered."

Several principles for construction of stable roads in western Oregon and northern California should be re-emphasized here. The first principle is "an ounce of prevention is worth a pound of cure"; avoid unstable terrain if at all possible. Utilize geologic maps, aerial photos and careful on-the-ground scouting to locate potential slope failures. Second, be prepared to sacrifice alignment, road width, and road grade to avoid trouble spots. Third, if problem areas must be traversed, be prepared to implement preventative measures during road construction. Fourth, maintenance activities should be geared to maintain or improve road stability. This will include keeping a close watch for symptoms of impending slope failure. Finally, for each activity concerned with the road system, such as road location, design, construction, and maintenance, utilize every opportunity to control water — both surface water and ground water.

We believe that if the information which is found in this publication is applied to the problems of stable road construction in western Oregon and northern California then the number of slope and road failures caused by "acts of man" can be reduced by as much as 50 percent as a practical goal.

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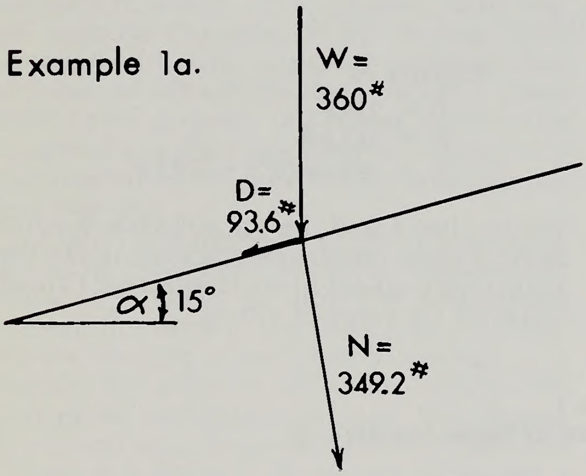
APPENDIX A

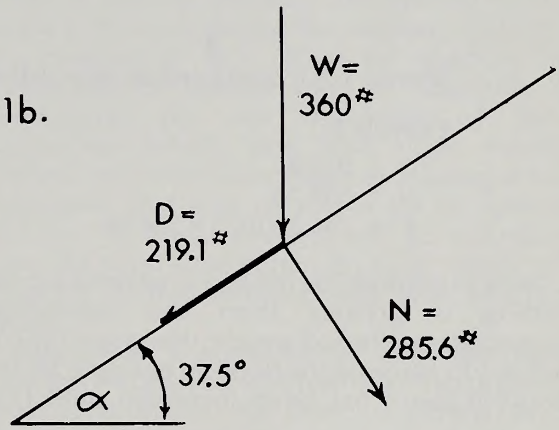
CALCULATION OF FRICTIONAL RESISTANCE TO SLIDING

To calculate the frictional resistance to sliding for a weight resting on a sloping surface, it is first necessary to understand how to calculate these components of the weight which act parallel and normal (perpendicular) to the surface. The magnitude of these components depends upon the weight of the soil and the slope gradient. The examples below show how these components are calculated for a column of soil 1 foot square and 3 feet deep which weighs 120 pounds per cubic foot. Sines and cosines of slope angles are given in Table I.

Example 1a.



1b.



Where: W = weight of soil column 1 foot square and 3 feet deep
= 1 foot^2 x 3 ft. x 120#/foot^3 = 360#.

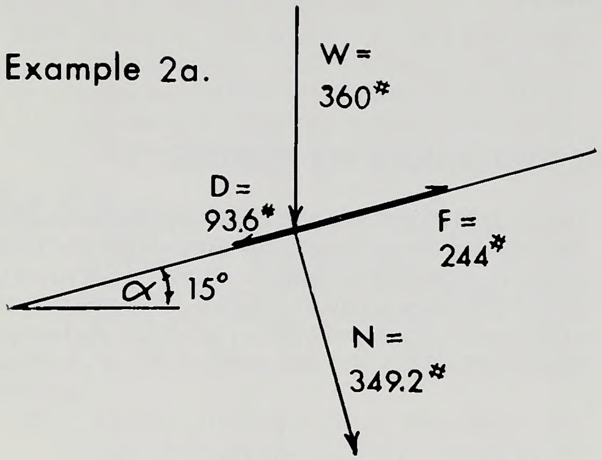
N = Component of the soil weight which acts normal (perpendicular) to the surface.

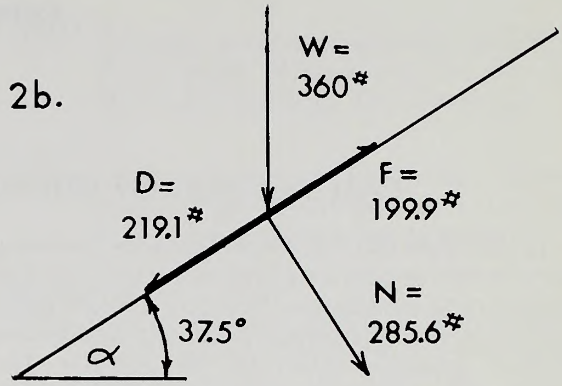
D = Downslope component of the soil weight.

Example 1a
Slope gradient, α = 15° or 27%
W = 360#
N = W (cos 15°)
= 360# (0.97) = 349.2#
D = W (sin 15°)
= 360# (0.26) = 93.6#

Example 1b
Slope gradient, α = 37.5° or 77%
W = 360#
N = W (cos 37.5°)
= 360# (0.79) = 285.6#
D = 360# (sin 37.5°)
= 360# (0.61) = 219.1#

The frictional resistance to sliding depends upon 1) the force which acts normal to the surface and 2) the coefficient of friction. The coefficient of friction is a physical factor which converts a portion of the normal force to frictional resistance to sliding. The frictional resistance to sliding acts parallel to the slope but in an opposite direction to the downslope component of the weight. Frictional resistance to sliding, F, is equal to the product of the normal force, N, and the coefficient of friction; as the normal force increases, the frictional resistance to sliding also increases. The coefficient of friction for sands ranges from about 0.53 for loose sands to about 1.00 for densely compacted sand and gravel. An average value of 0.7 for the coefficient of friction of medium dense sand will be used in the examples below.





Where: F = frictional resistance to sliding = N(0.7).

Example 2a

D = 93.6#
N = 349.2#
F = 349.2# (0.7) = 244.4#

Example 2b

D = 216#
N = 285.6#
F = 285.6# (0.7) = 199.9#

Note in example 2a, the frictional resistance to sliding is greater than the downslope component of the soil weight, therefore, the soil will stay in place on the slope. In example 2b, the slope gradient has been increased until D is greater than F and the soil will slide down the slope. Various other slope gradients may be tried to find the gradient where D will equal F and the soil is on the verge of sliding.

Table I
Sines, Cosines, and Tangents of Slope Gradient, ϕ

Slope Gradient (Percent)	Sin	Cos	Tan	Slope Gradient (Degrees)	Sin	Cos	Tan
0	0.00	1.00	0.00	0	0.00	1.00	0.00
5	0.05	1.00	0.05	2.5	0.04	1.00	0.04
10	0.10	1.00	0.10	5	0.09	1.00	0.09
15	0.15	0.99	0.15	7.5	0.13	0.99	0.13
20	0.20	0.98	0.20	10	0.17	0.98	0.18
25	0.24	0.97	0.25	12.5	0.22	0.98	0.22
30	0.29	0.96	0.30	15	0.26	0.97	0.27
35	0.33	0.94	0.35	17.5	0.30	0.95	0.32
40	0.37	0.93	0.40	20	0.34	0.94	0.36
45	0.41	0.91	0.45	22.5	0.38	0.92	0.41
50	0.45	0.89	0.50	25	0.42	0.91	0.47
55	0.48	0.88	0.55	27.5	0.46	0.89	0.52
60	0.51	0.86	0.60	30	0.50	0.87	0.58
65	0.54	0.84	0.65	32.5	0.54	0.84	0.64
70	0.57	0.82	0.70	35	0.57	0.82	0.70
75	0.60	0.80	0.75	37.5	0.61	0.79	0.77
80	0.62	0.78	0.80	40	0.64	0.77	0.84
85	0.65	0.76	0.85	42.5	0.68	0.74	0.92
90	0.67	0.74	0.90	45	0.71	0.71	1.00
95	0.69	0.72	0.95				
100	0.71	0.71	1.00				

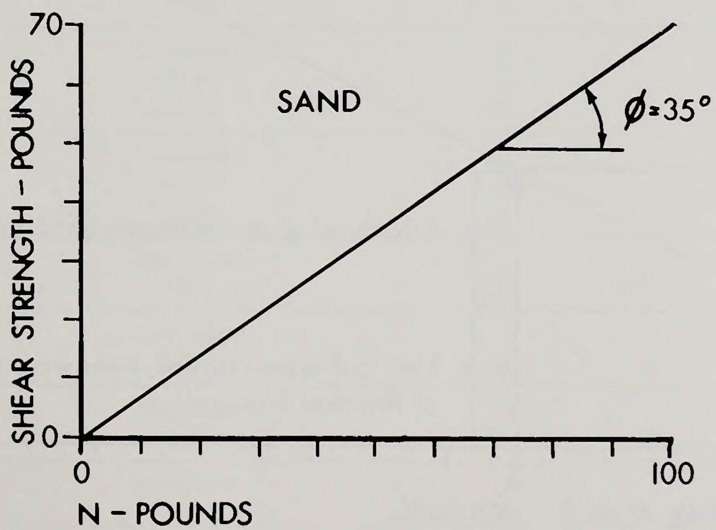
APPENDIX B

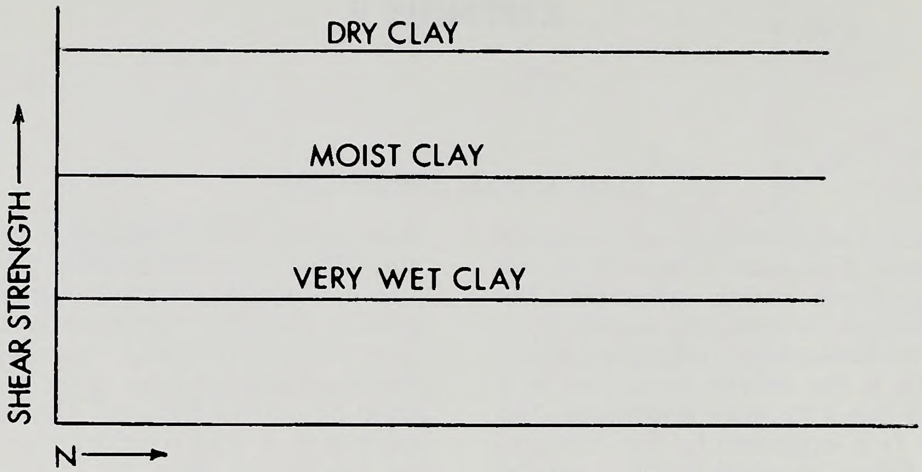
SOIL SHEAR STRENGTH

When a block of soil is sheared, one portion of the block moves past another portion as the result of two forces acting in parallel but opposite directions. In examples 2a and 2b in Appendix A, the downslope component of the soil weight (D) is the *driving force*, that is, it causes the soil block to move downslope. The driving force D is oppossed by the frictional resistance to sliding (F) which acts in a parallel but opposite direction. If the driving force exceeds the frictional resistance to sliding, then the block of soil will fail along the shear plane shown in the diagram. The driving force can be thought of as a *shear force* and the resistance to movement along the potential plane of failure as *shear strength.*

For sands and gravels whose shear strength is derived entirely from frictional resistance to sliding, it is easier to show the relationship between the normal force acting on a surface and the resulting shear strength by means of a diagram. The slope of the line is equal to the value of the coefficient of friction. The diagram on the next page shows the shear strength line inclined at a slope of 70 percent. Note that a shear strength of 70 pounds is generated by a normal force of 100 pounds. In much of the literature dealing with soil mechanics and slope stability the term coefficient of friction is not used. Instead, frictional resistance to sliding is expressed as a function of the tangent of "*the angle of internal friction.*" The symbol for this angle is ϕ, as shown on the diagram. Note that the tangent of $\phi = 35°$ is 0.7 which is the coefficient of friction used in Appendix A. Therefore, for the remainder of these appendices which deal with slope stability analysis, the frictional resistance to sliding will be expressed in terms of the angle of internal friction, ϕ. Shear strength will be calculated from the expression N (tan ϕ) or by means of a diagram. Either method will give identical results.

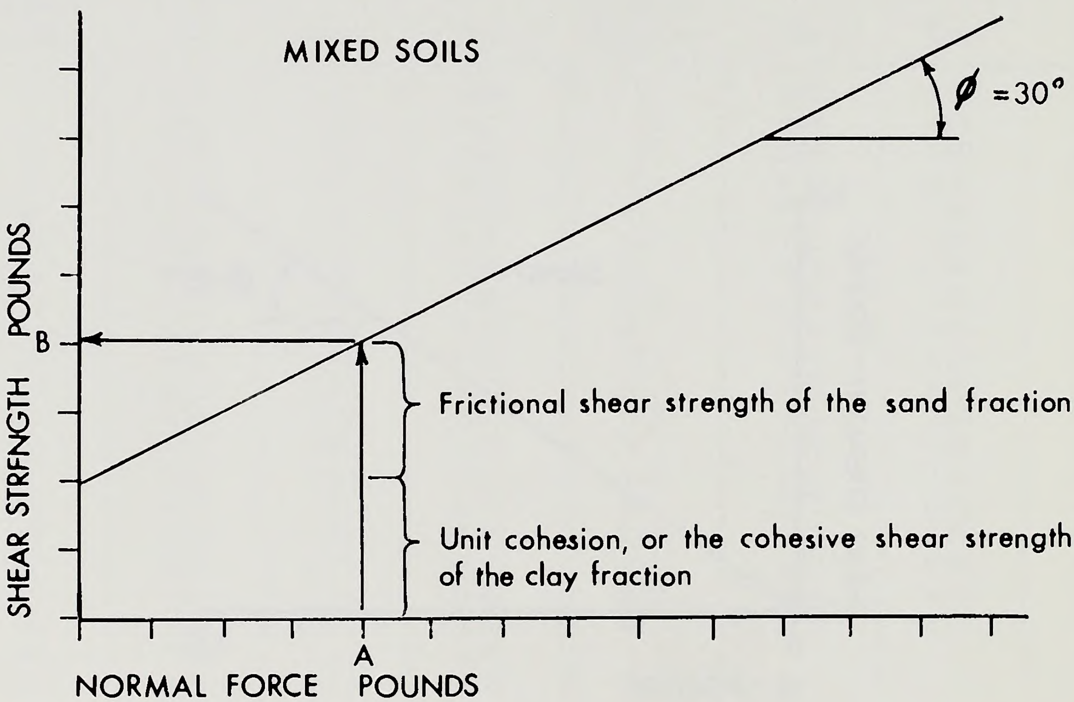
Clay adds a measure of *cohesive* shear strength through cohesion which is the property that causes soil particles to stick together. Two of the most important characteristics of cohesive shear strength are shown in the diagram below. First, the shear strength of pure clay decreases as the water content increases. Second, the normal





force has practically no influence on the shear strength of clays. Note that cohesive shear strength is expressed as pounds per square foot.

Forest soils are rarely pure sands or pure clays. They are *"mixed soils,"* that is, they contain sand, silt, and clay in varying proportions which affects the shear strength of the soil as shown in the next diagram. If the diagram is entered at a normal force of **A** pounds then the shear strength found at **B** is the sum of the cohesive shear strength of the clay fraction and the shear strength developed in the sand as a result of the normal force. In later appendices the total shear strength of a mixed soil may be determined by a diagram or by the numerical expression: Shear strength = C + N (tan ϕ). In this equation **C** is the unit cohesion and N (tan ϕ) is the frictional shear strength. This equation and the diagram give identical results.



APPENDIX C

DEBRIS AVALANCHES AND DEBRIS FLOWS

These types of mass failures move parallel to the shear plane. The most common portions of the road which may fail in this manner are sidecast material on steep slopes below the road and shallow soils over bedrock above the road cut. The plane of failure may be the contact between soil and smooth bedrock or, if the bedrock surface is rough, the plane of failure may be found within the soil, but parallel to the bedrock. Tree stumps incorporated within the fill material may serve to anchor the soil to the underlying rock and cause the plane of failure to be found within the fill material. If the anchoring tree roots decay after a few years, then the failure plane may again be found closer to the bedrock surface.

This appendix describes methods to evaluate the relative stability of masses of soil which tend to slide as debris avalanches and debris flows. Certain information is necessary before a stability analysis may be made: The weight of the material (pounds per cubic foot) when it is wet but not saturated; the saturated weight of the material; the angle of internal friction, ϕ. Soil scientists, engineers and geologists may be able to either measure these properties or to estimate values from those of closely related materials. Table II gives some estimates of ϕ angles for

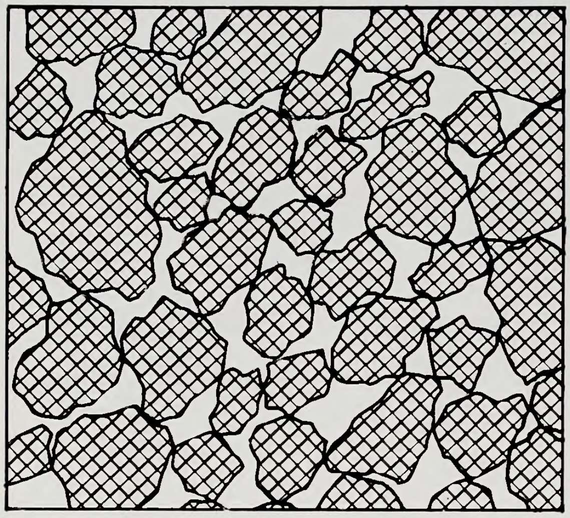
Table II
Typical Values of Friction Angles and Unit Weights for Various Soil Materials
(Adopted from B. K. Hough, BASIC SOILS ENGINEERING.
Copyright 1957. The Ronald Book Company, New York)

Classification	Density or Consistency	Friction Angle, in degrees	Unit Soil Weight Lbs. per Cubic Feet
Coarse Sand or Sand and Gravel	Compact	45	140
	Firm	38	120
	Loose	32	90
Medium Sand	Compact	40	130
	Firm	34	110
	Loose	30	90
Fine Sand	Compact	34	130
	Firm	30	100
	Loose	28	85
Fine Silty Sand or Sandy Silt	Compact	32	130
	Firm	30	100
	Loose	28	85
Fine Uniform Silt	Compact	30	135
	Firm	28	110
	Loose	26	85
Clay-Silt	Medium		120
	Soft	20	90
Silty-Clay	Medium		120
	Soft	15	90
Clay	Medium		120
	Soft	10	90
Clay	Medium		120
	Soft	0	90

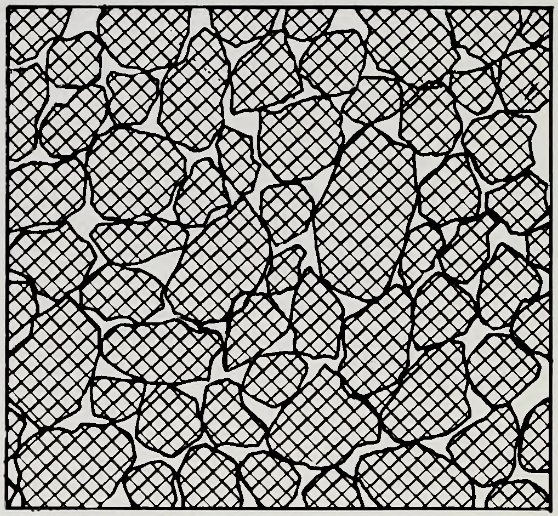
various materials at different densities and consistencies. The values should be used as rough estimates only, and are included here to show the variation between materials.

Note in Table II that both the unit weight and the friction angle are affected by the density and consistency of the material. Of even more importance to slope stability is the interaction between density and groundwater when the mass begins to shear along a failure plane. A representation of loose and compact arrangements of soil particles is shown below.

effective normal force and thereby reduces the shear strength. If loose, saturated, cohesionless materials are subjected to sudden shocks by earth tremors, nearby rock falls, etc., then the sudden increase in pore water pressure and loss of shear strength can result in *liquefaction* of the material. This illustrates two of the advantages of compacting fill material during road construction: it increases the density by reducing the pore space and it also increases shear strength by creating stronger interlocking of soil particles.



LOOSE

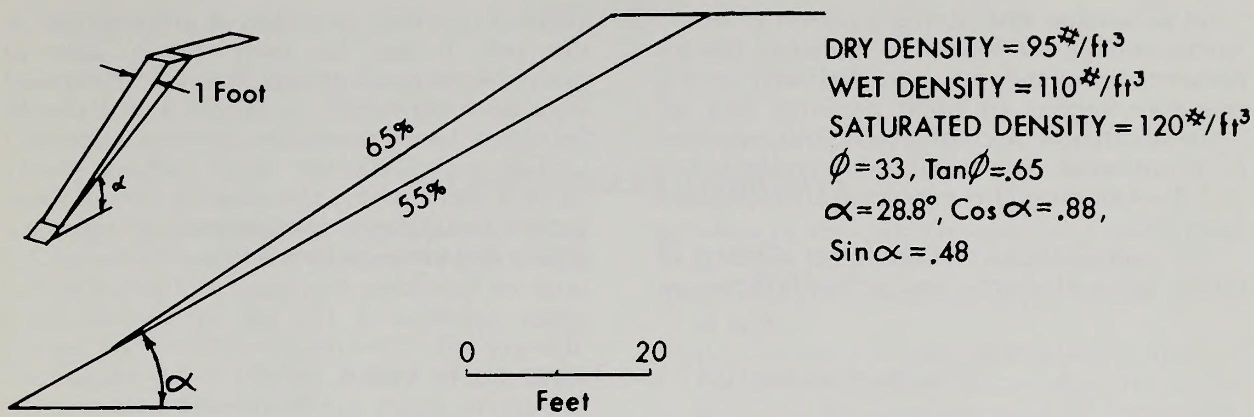


COMPACT

If each of these materials have their pore spaces filled with water, and is subjected to a normal and a shear force, there will be two different results. Particles in the compact material will tend to override each other as the material is sheared. This results in an expansion of the volume occupied by the material and a corresponding increase in the pore space. This in turn puts the pore water into tension because the volume of the pore space tends to increase but no more water is added. This combination of events increases the effective normal force with a comparable increase in the shear strength. By contrast, the loose material tends to become more compact as the soil particles shift position during shear. This creates a more compact density and reduces the pore space with a corresponding increase in the pressure on the water within the pores. This reduces the

The measure of the relative stability of a mass of soil is the "Factor of Safety" which is the ratio of those forces resisting sliding to those forces causing sliding: the factor of safety, *FS*, = *Forces resisting sliding/Forces causing sliding.* It can be seen from this equation that whenever the forces resisting sliding exceed those causing sliding then the factor of safety will be greater than one and the slope can be considered stable. Factors of safety of one or less indicate that the slope is either actively failing or is at the point of impending failure.

The diagram below shows a common source of debris avalanches and flows during road construction: a loose fill of essentially cohesionless sand and gravel. Estimated values of the following properties are shown to the right of the diagram. The dry density is the weight of the material as found in the summer



when soil moisture is lowest. The wet density is the weight of the material during an average day during the winter when some, but not all, of the pore space is filled with water. The saturated density is the weight of the material when all of the pores are filled with water. Note that the fill slope is 65 percent which is the tangent of the angle of internal friction. This so-called *angle of repose* is the maximum slope at which this material may be expected to stand.

In making an analysis of a slope such as this, only a section one foot wide is used and the diagram above represents the cross-sectional view of this one foot section.[1] The analysis assumes that the section is representative of conditions within the entire mass; if the section is stable, then the mass will be stable. The total weight of the section may be calculated by measuring the cross-sectional area of the fill section in square feet and multiplying this by the density of the material. The area may be measured by drawing the fill to scale and placing this over a sheet of graph paper on a light table and counting the squares in the fill section. The number of squares multiplied by the number of square feet per square gives the area in square feet.

The area of this fill is 185 square feet; the volume of this one foot section is therefore 185 cubic feet; and the wet weight of this volume is 185 ft.^{3} × 110 #/ft.^{3} = 20,350#.

Shear force = W (sin a) = 20,350 x 0.48 = 9,768#

Normal force = W (cos a) = 20,350 x 0.88 = 17,908#

Shear strength = N (tan ϕ) = 17,908 x 0.65 = 11,640#

$$\text{Factor of Safety} = \frac{\text{shear strength}}{\text{shear force}} = \frac{11{,}640}{9{,}768} = 1.19 \text{ stable}$$

[1]The procedure which follows illustrates the calculation of the factor of safety of the slope. There are numerous computer programs in use by government agencies and universities which can be used to make these calculations.

Let us assume that during a period of heavy winter rainfall, the lowest six inches of this fill becomes completely saturated with water and is therefore subject to uplift pressure. The fill material can now be considered to be composed of two materials — wet soil and saturated soil — and the weights of these volumes are calculated separately.

This analysis shows that the factor of safety of this fill material approaches perilously close to a value of one with six inches of groundwater in the soil. It can be seen that a layer of groundwater much greater than six inches could unbalance the stability of this fill. Also, it should be remembered that loose saturated material is subject to liquefaction when sudden shocks cause a slight shift in the mass. In view of these factors, this fill has a high potential for failure as a debris avalanche or debris flow.

Weight of saturated soil = 35 $ft.^3$ x 120 #/$ft.^3$ = 4,200#

Weight of wet soil = 150 $ft.^3$ x 110 #/$ft.^3$ = 16,500#

Total weight = 20,700#

Shear force = W (sin a) = 20,700 x 0.48 = 9,936#

Normal force = W (cos a) = 20,700 x 0.88 = 18,216#

Buoyant force = 62.4 #/$ft.^3$ x ½ ft. x 70.5 $ft.^2$ = 2,200#

Net normal force = Normal force - Buoyant force = 16,016#

Shear strength = N (tan ϕ) = 16,016 x 0.65 = 10,410#

$$\text{Factor of Safety} = \frac{10{,}410}{9{,}936} = 1.05$$

APPENDIX D

SLUMPS AND EARTHFLOWS

Slumps and earthflows are types of mass failures which occur in finer textured soils where cohesion may be a significant portion of the total shear strength of the soil. In addition, many slumps and earthflows exhibit a characteristic rotational type of failure, at least in the initial stages of mass movement. The rotational type of failure requires an entirely different type of slope stability analysis than that for debris avalanches.[1] The plane of failure is assumed to be a circular arc and a one foot section of the soil mass is divided into vertical slices. The analysis consists of summarizing the stability of the individual slices. Each slice is assumed to slide along the plane of failure to a point below the center of rotation. As shown in the diagram on the next page, those slices to the right of the center of rotation tend to cause movement of the total mass along the failure plane. Those slices to the left of the center of rotation tend to resist movement of the total mass. Note that the weight of each slice is represented by a vector which acts vertically through the center of gravity of each slice. The normal and downslope components of each weight vector are calculated as in earlier appendices and are drawn on the figure in the direction in which they act. Each normal component acts perpendicular to the plane of failure at the point where the line of the weight vector crosses the plane of failure. By convention, the downslope component is drawn in a direction tangential to the plane of failure where the weight vector crosses this plane, but for ease of illustration, the downslope components are shown attached to the end of the normal component. However, it should be remembered that the downslope component of the weight of each slice acts to cause a shear force along the failure plane. Opposing this shear force is the frictional resistance to sliding and the cohesive shear strength which also acts along the failure plane. The cohesive shear strength for each slice is the product of the unit cohesion and the contact area of the failure plane under the slice.

[1]*The procedure which follows illustrates the calculation of the factor of safety of the slope. There are numerous computer programs in use by government agencies and universities which can be used to make these calculations.*

Step 1: Draft slope and potential failure surface to scale.

Step 2: Divide mass into vertical slices and find the center of gravity of each slice. The center of gravity for each slice may be "eyeballed" in.

Step 3: Calculate the volume of each slice in cubic feet by measuring the surface area from the scale drawing. Remember that the section is one foot thick perpendicular to the plane of the paper (see sketch in Appendix C). If ground water is present, calculate the volume of the slice above and below the groundwater surface.

Step 4: Calculate the weight of each slice as the product of the volume of the slice and the density of the material. If ground water is present, calculate the weight of the slice above ground water using the wet density, and the weight below the ground water surface using the saturated density.

Step 5: Draw a vertical line from the center of gravity through the failure plane. Then scale the weight vector (W) of each slice along this vertical line beginning at the failure plane.

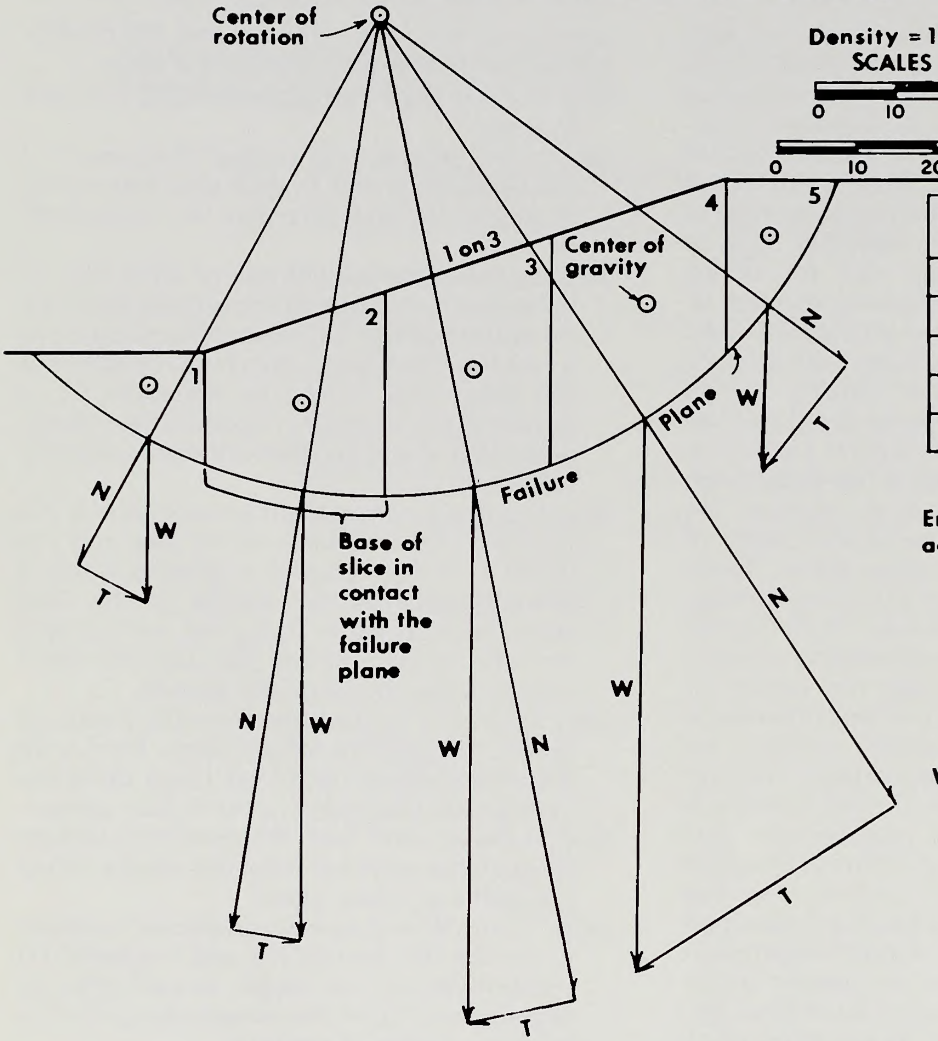
Step 6: Draw a line from the center of rotation through the intersection of the weight vector (W) and the failure plane.

Step 7: Use a 90° transparent draftsman's triangle to lay out the normal (N) and tangential (T) components of the weight of each slice by aligning one leg of the triangle along the line from the center of rotation.

Step 8: Scale the magnitude of each normal (N) and tangential (T) component using the same scale as for the weight vector (W). Enter these values in the table on the diagram paying careful attention to whether each tangential component causes sliding (T_d) or resists sliding (T_r).

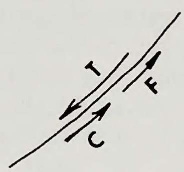
A more complete slope analysis of a potential rotational failure is shown in the next diagram. The 8 steps given above may be used to develop the various forces. Note that the tangential components of the weight correspond to the downslope components of weight used in earlier appendices. The tangential components

DIAGRAM OF CALCULATION AND LAYOUT OF COMPONENTS FOR THE STABILITY ANALYSIS OF ROTATIONAL SLUMPS



Slice No.	Volume Ft3	Weight
1	175	21,000
2	480	57,600
3	573	68,760
4	589	70,680
5	177	21,240

Enlarged view of forces acting on the failure plane

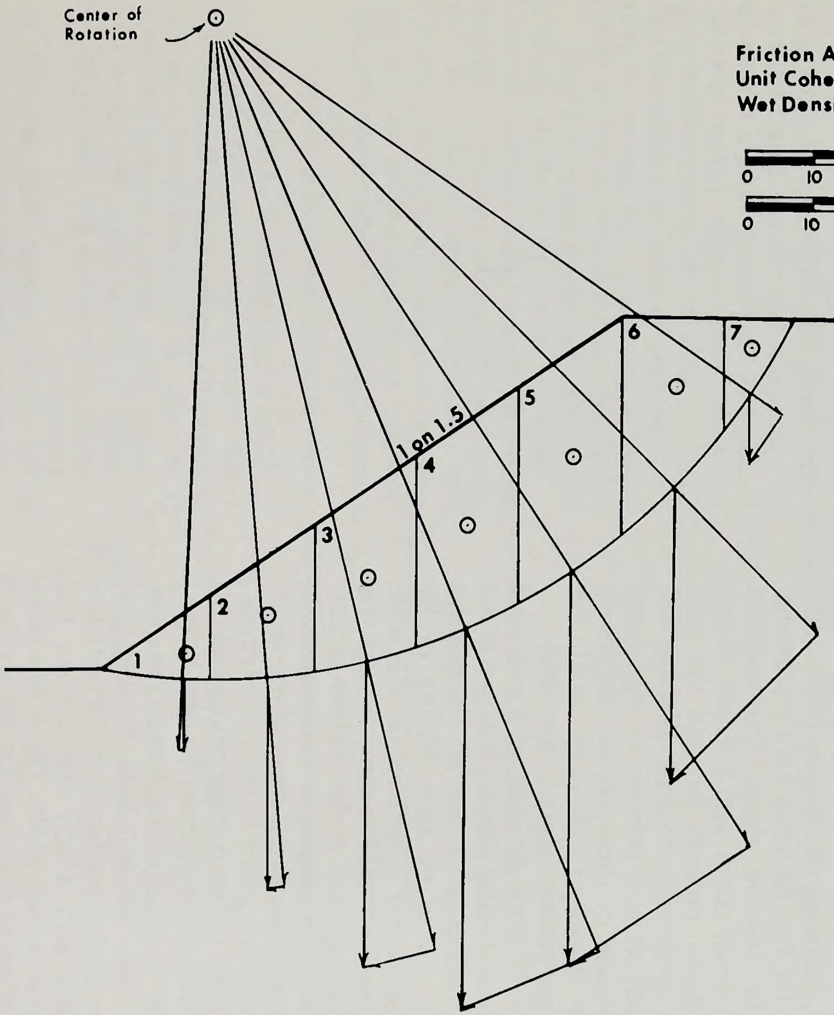


Where: T=Tangential component of soil weight
F=Frictional resistance to sliding: F=N·tanϕ
C=Cohesive shear strength

W=Weight of slice (Volume·Density). Scaled on vertical line from center of gravity.
N = Normal component of weight. Scaled from line through center of rotation.
T=Tangential component of weight. Scaled from line drawn 90° to N and touching W.

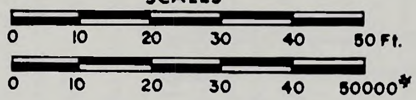
Adapted from B. K. Hough — BASIC SOILS ENGINEERING, Copyright 1957, The Ronald Press Company, New York.

SLOPE ANALYSIS OF A POTENTIAL ROTATIONAL FAILURE WITH NO GROUND WATER PRESENT USING SEMIGRAPHICAL METHODS



Friction Angle, $\phi = 15°$; $\tan\phi = 0.27$
Unit Cohesion, $C = 500\#/ft^2$
Wet Density $= 126\#/ft^3$

SCALES



Slice No.	Weight of Slice	Normal Force N	Tangential Forces-T Driving T_d	Tangential Forces-T Resisting T_r	Frictional Resist. $F = N(\tan\phi)$	Base Area A	Cohesive Shear Strength C·A
1	10,100	10,000		500	2,700	15.0	7,500
2	29,700	29,400	2,500		7,938	15.0	7,500
3	42,000	41,200	10,500		11,124	15.0	7,500
4	52,800	48,700	20,800		13,149	15.8	7,900
5	54,600	46,000	30,700		12,420	18.0	9,000
6	41,400	29,000	29,500		7,830	20.0	10,000
7	9,800	5,400	8,000		1,458	18.0	9,000
Totals			102,000	500	56,619		58,400

$$\text{Factor of Safety} = \frac{\Sigma C \cdot A + \Sigma N(\tan\phi) + \Sigma T_r}{\Sigma T_d}$$

$$= \frac{58{,}400 + 56{,}619 + 500}{102{,}000}$$

$$= \frac{115{,}519}{102{,}000} = 1.13 \text{ Stable}$$

Adapted from B. K. Hough — BASIC SOILS ENGINEERING, Copyright 1957, The Ronald Press Company, New York.

are divided into two groups: driving forces (T_d) that cause sliding, and resisting forces (T_r) that resist sliding. Frictional resistance to sliding is calculated as before. Cohesive shear strength is calculated by first measuring the length of the failure plane under each slice, and converting this to contact area by the fact that the section through the potential slump is one foot thick. The unit cohesion in pounds per square foot multiplied by the contact area in square feet yields cohesive shear strength in pounds. The factor of safety uses the totals (indicated by the symbol Σ in the equation) for cohesive shear strength, frictional resistance, and the driving and resisting tangential components.

The slope analysis in the last example did not include the effect of ground water. A complete analysis of any potential rotational failure must be based on detailed information of the types of materials present, the thickness and orientation of layers of clays, shale, etc., and the location of the ground water table. This information is usually derived from core drilling. In Appendix C the ground water occupied only a few inches of the bottom of the loose cohesionless fill and usually moves quite rapidly through this sandy, gravelly material. On the other hand, rotational slumps usually involve deep, fine-textured material with small pores and a much slower velocity for water moving through the soil. Under these conditions the ground water will occupy a much greater portion of the slump mass and, because of these greater depths of water, the buoyant forces will be correspondingly greater in clays than sands.

An example of slope analysis of a potential slump with ground water present is shown in the next two diagrams (pages 97 and 98). The ground water level is found by core drilling or by observation wells and is located on the scale drawing of a one foot thick section through the slope. The development of the weight vector, and the normal and tangential components is done according to the first 8 steps of the semi-graphical procedure given in the last example. The development of the buoyant force and the effective normal force is given in the next 5 steps.

Step 9: Use a 90° transparent draftsman's triangle to draw a line which passes through the intersection of the weight vector (W) with the failure plane and which also crosses the ground water surface at 90°. This is line ab in the small enlarged drawings. Do this for each slice.

Step 10: Project each line ab onto a vertical line through the center of gravity of each slice. This line represents the vertical head of water which causes the buoyant force and is labeled cb on the enlarged drawing on page 98.

Step 11: Calculate the buoyant force for each vertical slice as the product of the scale length of line cb (in feet) times 62.4 #/ft.^{3} times the contact area under each slice in square feet: Buoyant Force = line cb (ft.) x 62.4 #/ft.^{3} x Contact Area (ft.^{2}). Note, in the case of slice 6 where ground water does not extend over the entire base of the slice, use an average value of the vertical head acting over that portion of the contact area which is affected by ground water.

Step 12: Using the weight scale, convert each buoyant force to corresponding length and lay this over each normal component of weight beginning at the failure plane.

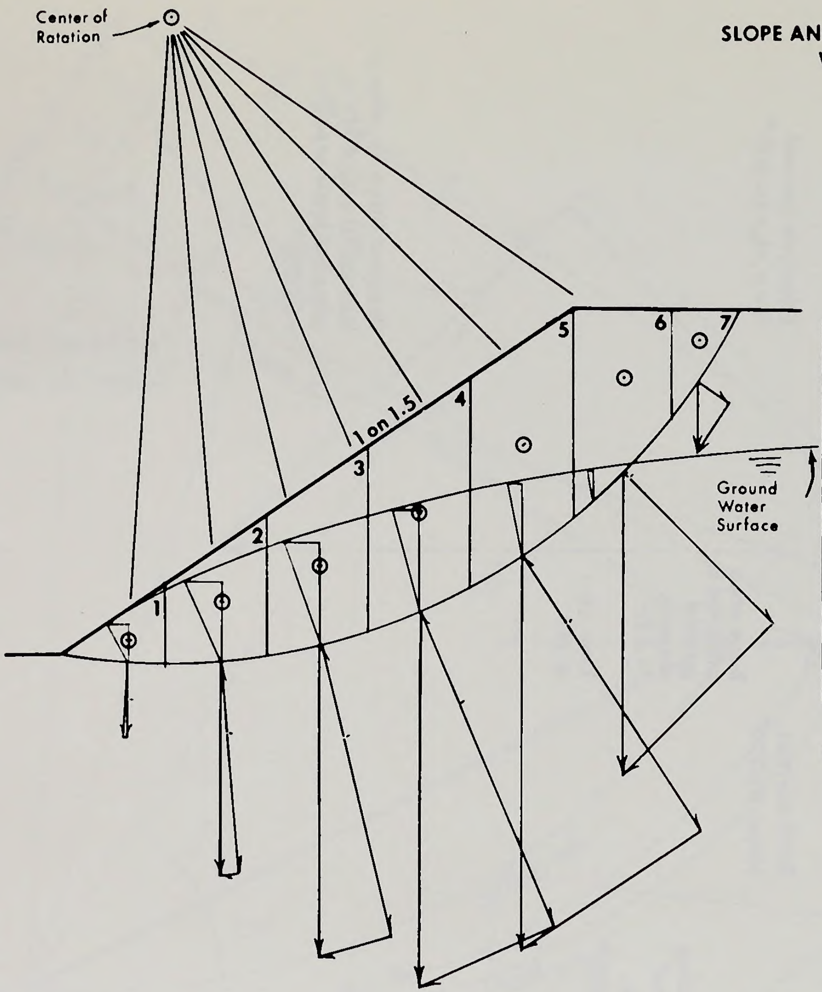
Step 13: The difference in length between each buoyant force vector and the normal force vector represents the effective normal force acting on the failure plane. Measure each effective normal force vector using the weight scale and enter the effective normal force in the table on the diagram.

The foregoing examples of stability analyses for rotational failures are based upon a given location for the plane of failure. It is obvious that there are innumerable failure planes which could be located on a cross section of a slope. The complete analysis of a potential rotational failure is actually a trial and error process of analyzing various planes of failure to find that plane with the lowest factor of safety

There are several rules which can be used to locate the most likely location of the failure plane and reduce the number of trials to locate the so-called "worst circle." For relatively uniform materials, a first trial for the worst circle may be located according to the graphical construction given in the figure on page 99.

This procedure is used as follows:

1. Make a scale drawing of the slope.
2. For slope angle, α, select the appropriate values for angles i and β. Draw a line inclined at angle β from an extension of the upper slope, then draw a second line inclined at angle i from the slope. The center of the trial circle is located at the intersection of these two lines.
3. For slopes steeper than about 1 on 2 (50 percent or 26.6°) the trial circle usually passes through the toe of the slope as shown in the diagram above. For flatter slopes, the trial circle will intersect the slope below the toe;



SLOPE ANALYSIS OF A POTENTIAL ROTATIONAL FAILURE WITH GROUND WATER PRESENT USING SEMIGRAPHICAL METHODS

Friction Angle, ϕ=15°; Tanϕ = 0.27
Unit Cohesion, C=500#/ft²
Wet Density=126#/ft³
Saturated Density=132#/ft³

SCALES

0 10 20 30 40 50 Ft.

0 10 20 30 40 50000#

Slice No.	Weight of Slice	Effective Normal Force N_e	Tangential Forces T Driving T_d	Tangential Forces T Resisting T_r	Frictional Resistance $F=N_e(\tan\phi)$	Base Area A	Cohesive Shear Strength C·A
1	10,530	5,200		700	1,404	15.0	7,500
2	30,400	20,000	3,000		5,400	15.0	7,500
3	44,000	29,000	11,000		7,830	15.0	7,500
4	53,200	34,200	21,200		9,234	15.8	7,900
5	55,600	34,700	30,800		9,369	18.0	9,000
6	42,700	26,800	30,500		7,236	20.0	10,000
7	9,800	5,500	8,000		1,485	18.0	9,000
Totals			104,500	700	41,958		58,400

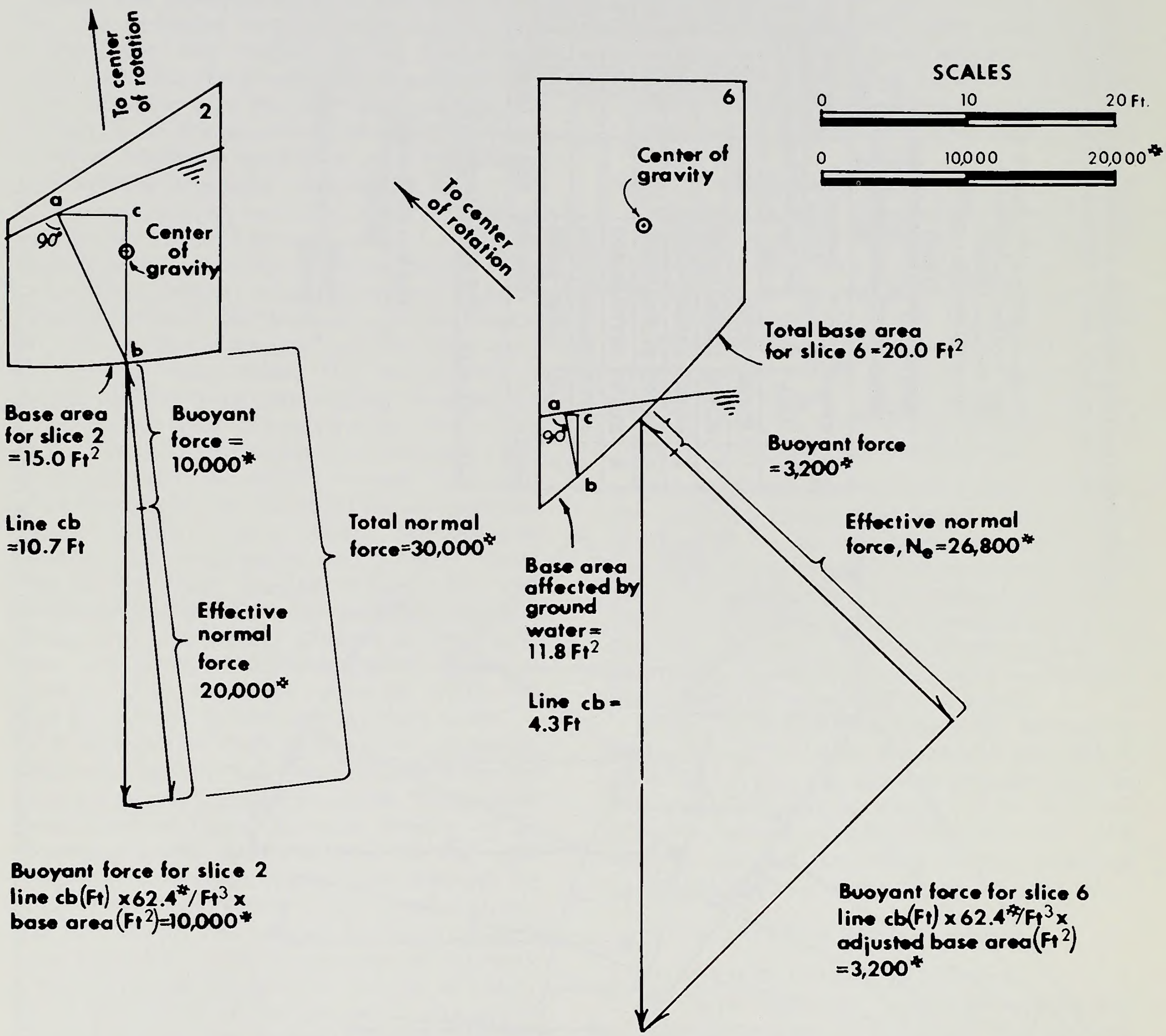
$$\text{Factor of Safety} = \frac{\Sigma C\cdot A + \Sigma N_e(\tan\phi) + \Sigma T_r}{\Sigma T_d}$$

$$= \frac{58{,}400 + 41{,}958 + 700}{104{,}500}$$

$$= \frac{101{,}058}{104{,}500} = 0.97 \text{ Unstable}$$

Adapted from B. K. Hough — BASIC SOILS ENGINEERING, Copyright 1957, The Ronald Press Company, New York.

ENLARGED DRAWINGS OF SLICES 2 AND 6 SHOWING SEMIGRAPHICAL CALCULATION OF BUOYANT FORCES



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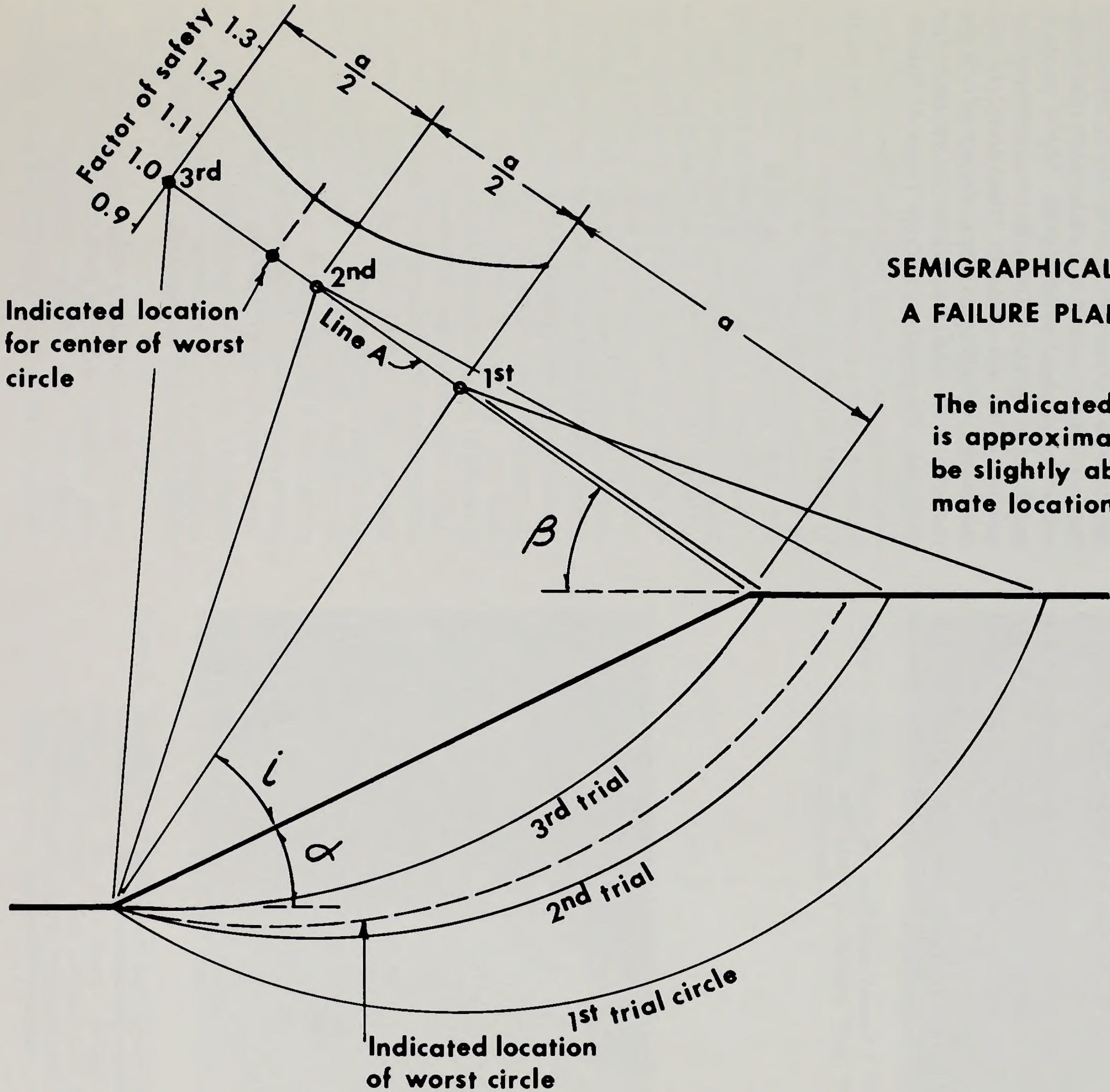
SEMIGRAPHICAL PROCEDURE FOR LOCATING A FAILURE PLANE IN ROTATIONAL SLUMPS



The indicated center of the worst circle is approximate. The actual location may be slightly above or below the approximate location.

Values for locating center of first trial circle

Slope	α	i	β
1 on 0.5	63°26′	38°	40°
1 on 1	45°	35°	37°
1 on 1.5	33°47′	32°	35°
1 on 2	26°34′	30°	35°
1 on 3	18°26′	30°	35°
1 on 5	11°19′	30°	37°

Adapted from B. K. Hough — BASIC SOILS ENGINEERING, Copyright 1957. The Ronald Press Company, New York.

the flatter the slope, the greater the curvature of the circle and the deeper the circle will dip into the base of the slope. If core drilling shows that there is a soft clay layer relatively deep within the slope, then the worst circle will probably pass through the upper part of this layer. If the soft clay is relatively shallow, then the failure plane may no longer be circular, but may follow the clay layer for some distance before breaking away to the surface. If it is suspected that a clay layer may distort the circular failure plane, then trials may be run using failure planes that pass along the clay layer to find the one with the lowest factor of safety.

4. After establishing a trial circle, proceed with the stability analysis and calculate a factor of safety.
5. Plot the value of the factor of safety on a scale which is at 90° to line A inclined at the angle β from the upper slope as shown in the diagram.
6. The center of the second trial circle is plotted on an extension of the base line through the first trial center but a distance a/2 from the first center. The distance, a, is the distance from the first trial center to the break in the upper slope as shown in the diagram.
7. Calculate a factor of safety for the new trial circle and plot this value as in step 5.
8. Find a third trial circle and repeat steps 4 and 5.
9. Connect the values of the factors of safety with a smooth curve and interpolate to find the lowest factor of safety. This will indicate the location of the center of rotation for the worst circle.

The accuracy of these stability analyses depends to a great extent upon the accuracy of the information about the underlying materials, including ground water levels. These analysis procedures will give best results whenever the soil materials are uniform. Most standard soil mechanics texts discuss slope analysis where the material is layered and where a clay layer has distorted the assumed circular plane of failure.

APPENDIX E

CONTROLLED BLASTING

A critical evaluation of blasting procedures must be made whenever constructing a road in hard bedrock on steep terrain. Often excavated rock not useable within the road design as fill or surfacing is disposed of by sidecasting. This is commonly achieved by overloading the rock formation with explosives so when fired the material would be thrown downslope, thereby eliminating much of the need for equipment handling of the remaining material. Such an operation creates havoc with watershed values (Figures 1 and 2). It buries productive soils, destroys timber and initiates debris slides and debris avalanches in drainageways which either destroy or severely impair stream habitats for both terrestrial and aquatic life.

National and State environmental standards stress the need to control as much as possible the excavated waste generated in road construction in a manner commensurate with the intent of those standards. An environmental objective of "controlled blasting" may simply be the prevention of material being lost downslope.

Many controlled blasting techniques are used to reduce overbreak of rock material and allow the material to generally retain its natural in-place position; thus making it much easier to endhaul to safe areas. These techniques all have one common method as expressed in Du Pont's Blasters' Handbook and that is the reduction and better distribution of explosive charges to minimize stressing and fracturing of rock beyond the designed excavation area. In all cases the Blasters' Handbook recommends *"That conservative trials be conducted to determine whether there is application (of chosen techniques) and, if so, then to establish optimum loads and patterns."* This especially holds true when encountering thinly bedded sediments and foliated metamorphic rock formations where bedding planes most often occupy some unknown orientation. In such instances overbreak can occur unless drilling can be done perpendicular, or nearly so, to the planes of the formation.

Engineers must realize that when they choose a blasting technique they are often dealing with geologic variables which are inherently complex. A relatively simple example would be

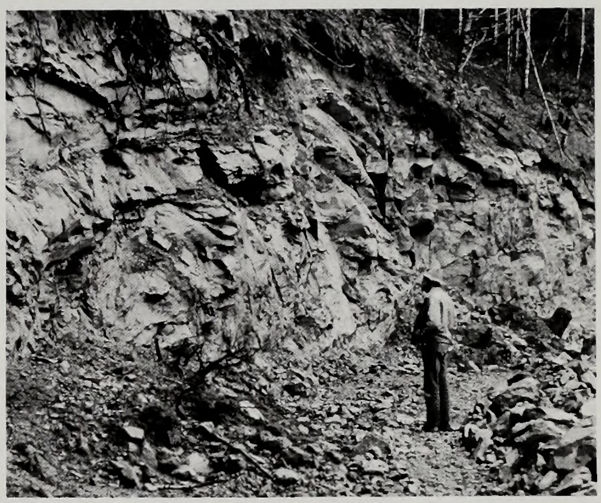


Figure 1. This shows the overbreak caused by random drilling and excessive charges. Note the very shallow soil mantle.

Figure 2. This photo was taken in the same area as Figure 1. It shows the results of overloading the production holes and the consequent sidecasting of excavated material. This sidecast material has destroyed valuable young Douglas fir and created a severe erosion and landslide hazard.

to use one blasting technique and expect it to be as equally successful in igneous formations as in rhythmically bedded sedimentary deposits. An intense materials investigation would reduce trial applications to establish optimum loads and patterns. Such an investigation may involve use of geophysical or drilling equipment and both can provide a distinct advantage in choosing a technique or combinations thereof to accomplish the purpose. Although in the absence of sound data on the rock structure and conditions of rock weathering, road design in bedrock requires judgement and experience with the material in the vicinity of the project.

Controlled blasting techniques most commonly used in construction of roads are briefly described as follows:

Pre-splitting (Pre-shearing)[1]

Pre-splitting basically is used to 1) produce a neat smooth backslope, 2) prevent overbreakage beyond the neat excavation lines and therefore reduce the amount of material that must be removed, and 3) prevent the shock force of the main blast from disturbing the material beyond the neat excavation lines, thus producing a more stable cut slope (Figure 3).

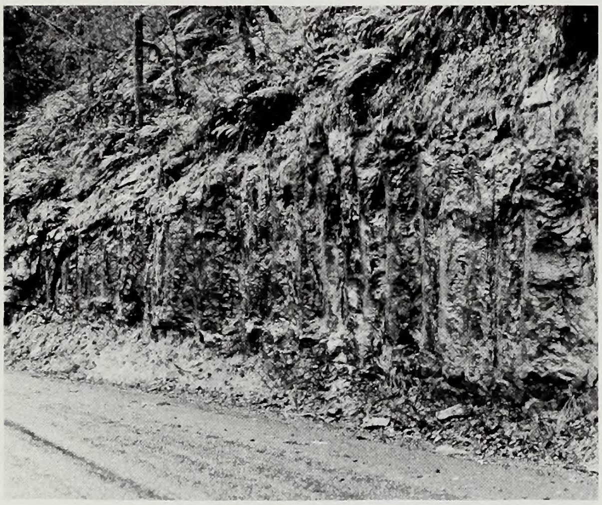


Figure 3. A pre-split cut slope. The drilling pattern was in-line with a 36-inch hole spacing. Each hole was loaded. The result is a relatively smooth cut slope with a minimum amount of sidecast.

Pre-splitting involves drilling a single row of vertical or near vertical holes along the neat excavation lines. These pre-split holes may be drilled and shot prior to drilling in the area to be excavated, or all holes may be drilled and the pre-split holes may be shot first using delays for the rest. Usually pre-split holes are drilled at a spacing of 30-50% of the normal distance between drill holes and usually all holes are loaded, although with a much reduced charge. This technique operates on the theory that when charges are fired simultaneously in adjacent holes, the zone between the holes will be fractured along a more or less smooth line connecting the holes as resultant of tension and shear forces. This fracture line (pre-split line) will then reflect or stop some of the shock waves from the production blasting that follows and will thereby minimize shattering and overbreak beyond the neat cut slope. Except with experienced drilling crews, the holes are drilled to a shallow depth in order to maintain alignment.

The variables of this technique are 1) drill hole diameter, 2) drill hole spacing, 3) distributions of charge, 4) pounds of explosives per foot of drill hole, and 5) explosive strength. Generally the explosive charge will be in the range of 0.10 to 0.40 pounds of explosive per square foot of area measured on the plane of the drill holes.

Controlled Production Blasting[2]

The objective of controlled production blasting is to adequately fracture the material so that it can be handled and yet hold the material within an area from which it can be removed with little or no damage outside of the excavated area. This technique may be used with or without pre-splitting.

The variables of this technique are 1) drill hole diameter, 2) drill hole spacing, 3) distribution of charge, 4) pounds of explosive per foot of drill hole and 5) explosive strength. Generally, the explosive charge will be in the range of 0.25 to 0.75 pounds of explosive per cubic yard of material

It should be noted that these techniques are very complicated and this appendix covers the subject only briefly and in general terms. It is recommended that a competent explosives specialist be consulted about the practice of these techniques.

[1]*Provided by Mr. Ronald Van Domelen, Project Engineer, Bureau of Land Management, Eugene, Oregon.*

[2]*Ibid.*

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